REPORT

Volume 1: Technical Report

Hydrodynamic modelling of Kalpasar closure dam

Client:	India Ministry of Earth Sciences, National Centre for Coastal Research (NCCR)
Reference:	BI5332-RHD-ZZ-XX-RP-Z-0001
Status:	Final/01
Date:	3 February 2023

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Document title: Volume 1: Technical Report

Subtitle: Hydrodynamic modelling of Kalpasar closure dam
Reference: BI5332-RHD-ZZ-XX-RP-Z-0001
Status: 01/Final
Date: 3 February 2023
Project name: Kalpasar Dam
Project number: BI5332-101-100
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Date / initials: 2 February 2023

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Date / initials: 3 February 2023

Classification
Project related

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1 Introduction

1.1 Context of the project

Identified first in the 1970's for its tidal power resource and having been considered in different configurations over the years, at present the KALPASAR dam aims to provide a reliable source of fresh water for the drought-stricken state of Gujarat (Saurashtra in particular) by means of a 60 km dam between Bhavnagar and Aladar of which the centre 30km concerns a closure dam in the Gulf of Khambhat . A reliable source of fresh water is establsihed once the closure dam is constructed and the transition from sea water reservoir to fresh water reservoir has occurred. From earlier studies it was found that the construction of the closure dam and the saltwater transition will require 10 to 15 years before fresh water is available, pending further analysis beyond the present scope of this assignment.

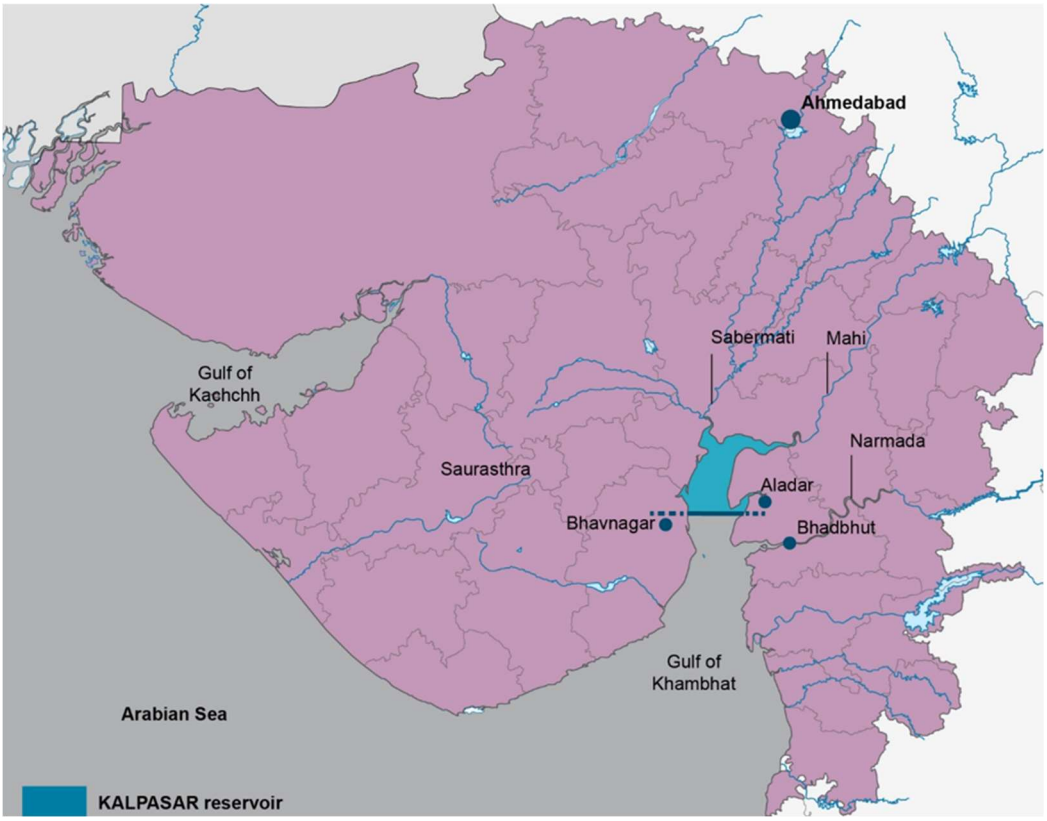


Figure 1-1: Selected alignment of the KALPASAR dam in the Gulf of Khambhat

In its present form, the KALPASAR project includes the 60 km dam connecting the Bhavnagar and Bharuch districts and a road/rail corridor, a freshwater reservoir of 7.800 million m^3 and a spillway to divert floods from the three major rivers flowing into the freshwater reservoir (Sabarmati, Mahi and Dhadhar). Once completed, a diversion from the Bhadbhut Barrage in the Narmada by means of a canal towards the freshwater reservoir is also considered by Narmada Water Resources, Water Supply and KALPASAR Department (NWRWS).

The KALPASAR dam is a huge undertaking considering the sheer volume of work and associated cost alone. Considering that during construction, (i) site conditions of more than 25 m water depth on average with poor soil conditions, (ii) a 4 m to over 10 m tidal range at Bhavnagar; and (iii) monsoon seasons that take out 3 months of working time each year must be accounted for amongst others, makes the project even more challenging and costly. A delay in its completion – if construction risks are not appropriately mitigated

– is likely to happen. Given the anticipated high cost of construction, the risk of delay before fresh water becomes finally available results in a viability gap that is difficult to mitigate, if at all. As such, KALPASAR dam is an unprecedented project.

The construction of the KALPASAR dam consists of two main stages: (i) the construction of the closure dam and (ii) the upgrading of the closure dam to the final (permanent) dam. The present feasibility study considers the feasibility of the closure dam only, seeking to identify an envelope of closure strategies with optimised construction risks. This is the first and probably most important step in de-risking KALPASAR project. After all, once the closure dam is erected, the construction risk associated with upgrading it to its desired final form is dramatically lower, although still significant for its size.

1.2 Overview of scope of assignment

The scope of the present assignment consists of two aspects, namely:

1. Design of Closure Methodology for KALPASAR Dam, comprising:
    a. Review of existing closure methodology reports and study various options for closure of KALPASAR dam; and
    b. Arrive at optimal closure methodology and design the same.
2. Design of gate and hydraulic mechanical system.

This scope has been executed and documented as described in the following sub-chapter.

1.3 Guidance for the Reader

The execution of the study is documented in a Detailed Project Report with four supplementary Technical Volumes as annexures, structured as follows:

Detailed Project Report (DPR)

- The Detailed Project Report (DPR) takes the results from the technical report on the KALPASAR closure dam feasibility (Volume 2) to assess and evaluate risks to account for when considering the alternatives identified. Risks addressed are amongst others volume risks, price risks, time related risks and the failure mode analysis as (key) technical risks and mitigated in their respective designs. The DPR includes an executive summary, key issues and risks to consider and recommendations on the way forward.

Volume 1: Technical Report on Hydrodynamic Modelling of KALPASAR Dam Closure **(THIS REPORT)**

- Volume 1 introduces the closure methods and strategies, and describes the hydraulic analysis based on thousands of simulations with the one-dimensional model hydraulic model, conducted to determine current velocities occurring during the closure for different closure options.

Volume 2: Technical Report on KALPASAR Closure Dam Feasibility Design

- From a thorough analysis of site conditions, a design basis for KALPASAR closure dam is determined to provide an unequivocal and solid basis for (i) development of distinct and feasible alternatives and (ii) risks to account for in the feasibility analysis presented in the Detailed Project Report. Gradual and sudden closure components are designed according to the design basis and used as building blocks allowing for evaluation of different closure scenarios in a construction model that provides insight into cost and duration of construction. From the evaluation of scenarios, the distinct closure strategies are derived and evaluated for input to the feasibility analysis. The construction model used results in a theoretical design that is subsequently translated into a practical design for (i) illustrative purposes and (ii) to serve as a basis for the Bill of Quantities and Cost Estimates in Volume 4.

Volume 3: Technical Report on Design Review of Permanent Structures KALPASAR Dam and Design of Spillway Gates

- Volume 3 contains the identification and assessment of options for the spillway gates, concluding in a conceptual design of the preferred option, a cost estimate and an operations and maintenance plan. In addition, a review of the KALPASAR dam structures in their permanent form, including the spillway, is provided as additional feedback on the designs by other consultants provided by the client.

Volume 4: Technical Report on Cost Estimate for KALPASAR Closure Dam

- Volume 4 contains specifications, a bill of quantities and an associated cost estimate for the KALPASAR closure dam elements.

The report at hand has the following structure. In Chapter 2 the closure methods and modelling approach are discussed. This chapter also contains the consideration of different numerical flow models for this study. In this study the 1-D flow model SOBEK is used. The theoretical background of SOBEK is explained in Chapter 3. The model set-up is explained in Chapter 4, and Chapter 5 focusses on the calibration of the model. Chapter 6 contains the results of the initial modelling. The case specific settings are also part of this chapter rather than being part of the Chapter 4: Model setup. This is because the case specific settings of each step are based on the findings of the previous step. The conclusions of this study are presented in Chapter 7. Finally, Chapter 8 presents the main conclusions, and explains the limitations and recommendations of the flow study.

2 Approach

In this chapter, the possible methods to close the estuary are discussed, followed by the discussion of analysis methods.

The first step in the construction of the Kalpasar Dam is the closure of the estuary. After the estuary is closed, the final dam needs to be constructed over the closure dam. When the construction of this dam is progressing, the cross-section over which the water can enter the estuary becomes gradually smaller. This results in larger flow velocities over the cross-section and an increase in risk of erosion. Therefore, larger stones are required to construct the next part of the closure dam. Larger stones are more difficult to mine and transport and are therefore more expensive. Minimizing the required stone sizes, in other words, minimizing the maximum flow velocity is an effective design goal. In other words, the maximum flow velocity and the corresponding minimum required stone size during the closure are important design criteria.

The flow velocities during the closure of the estuary are determined using a hydrodynamic (flow) model. Many flow models exist with each having their pros and cons. Some flow models are basic and have a limited representation of reality. On the other hand, these models have a very low computational effort and therefore many scenarios can be considered within limited time. More complex flow models have a better representation of the physical processes but also require more computational time and therefore less closure scenarios can be considered. The flow model has been selected based on the ideal balance between representation of the physical processes and the computational effort.

2.1 Closure methods

Multiple closure methodologies exist with each method having its own pros and cons. In this assignment they are sorted in 3 categories: horizontal closure, vertical closure and sudden closure. A horizontal closure is shown in the left panel of Figure 2-1. The gap is closed from the sides. This method has the advantage that limited water-based equipment is required and it can be built by dump trucks. On the other hand, the required stone size increases when the gap becomes smaller and smaller due to the increasing flow velocity.

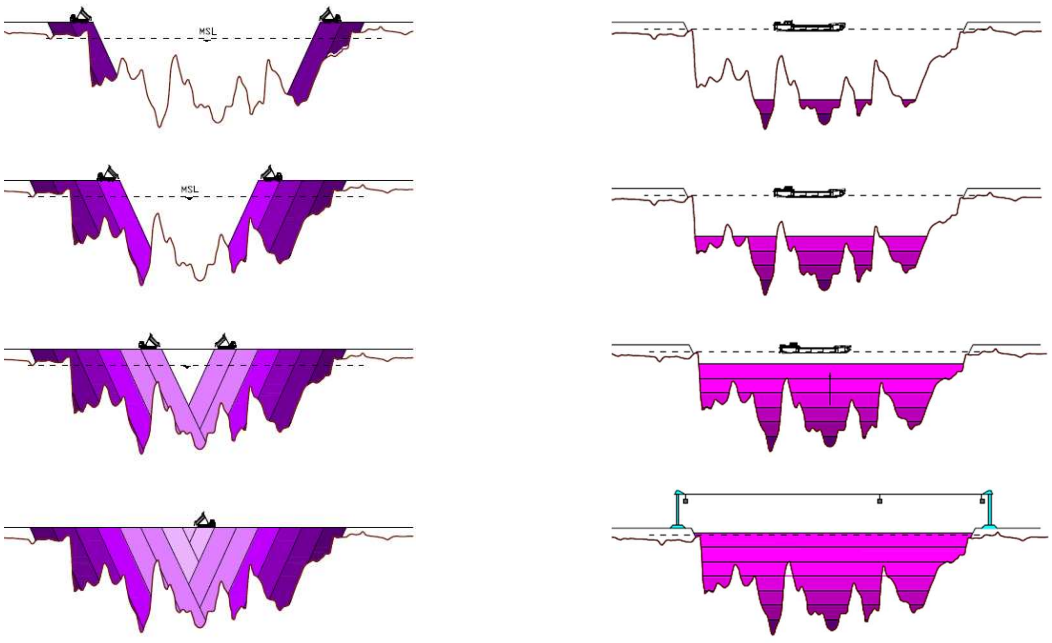


Figure 2-1: Left) an illustration of a horizontal closure. Right) an illustration of a vertical closure.

A vertical closure is presented in the right panel of Figure 2-1. Sand and rocks are placed horizontally over the entire gap, layer by layer. With this method mainly water-based equipment can be used. The advantage of this method is that maximum flow velocities during the closure are lower compared to a horizontal closure. Due to the smaller maximum flow velocity, the minimum required stone sizes for the horizontal closure are much smaller than for the vertical closure. Smaller stones are easier to mine and transport.

The third method is a sudden closure, see Figure 2-2. During a sudden closure large concrete elements called caissons are placed in the opening. Typical dimensions of these caissons are (100 m (l) x 10 m (w) x 25 m (h). The maximum flow velocity during a sudden closure is even smaller than for a vertical closure., However, caissons need to be made out of concrete, which can be expensive. Additionally, the caissons need to be accurately placed and this can be a challenging operation considering the strong tidal currents during the closure operation.



Figure 2-2: The sudden closure of the Brouwers dam, the Netherlands. Left) the transportation of the open caissons towards the gap. Right) the placing of the final caisson.

Two types of caissons can be used: open or closed caissons. Closed caissons as the name suggests are closed on all sides and the water cannot flow through them. They are placed in the in the opening and can be interpreted as extremely large rocks. An example of such a closure is given in Figure 2-3. The top view shows the difficulty of placing these caissons. Closed caissons need to be placed in the opening as the last part of the closure.



Figure 2-3: The emergency closure of Walcheren the Netherlands. In this specific case closed caissons were used because of the extreme flow velocities during the closure. Additionally, these caisson were "leftovers" of the Melburry harbours in Normandy constructed during the World War II.

Open caissons are open at the front and back and the water flows freely through them. The open caissons can be placed in the gap and will have limited effect on the flow conditions because the water will flow through them. When all the caissons are placed, all the open caissons are closed in a matter of hours using bulkheads. Open caissons do not need to be placed in the opening at the final phase of the closure. They can also be placed at the start of the closure of halfway during the closure. When the open caissons are placed in a later stage of the closure, it becomes more difficult to place them because the flow velocity have already increased.

In this study, combinations of vertical closure, horizontal closure and open caissons are considered. The different closure procedures will be variants of a typical closure procedure. In the typical closure procedure, the open caissons are placed first, after that the cross-section is party closed horizontally and after that vertically. The different closure procedures differ in length and position of the caissons, length of the horizontal closure and length of the vertical closure. In our scenarios the caissons are placed in step one because of the extreme flow conditions. After that the dam is built partly horizontally because in that stage still relatively small material can be used. We assume that a horizontal closure is symmetric i.e. evenly distributed between both sides. During the horizontal closure material can still be transported by trucks. When the stones become too large, the dam is constructed vertically to reduce the maximum flow velocity during the closure. We assume that the vertical closure is homogeneous over the vertical as shown in the right panel of Figure 2-1

2.2 Flow models

Three flow models have been considered for modelling of the flow velocity during the construction of the closure dam: a 0-D storage model [1], a 1-D model (SOBEK) [10] and a 2-D model (Delft3D) [3].

Storage model
An example of a storage model is displayed in Figure 2-4. It consists of multiple reservoirs which are connected with each other. The estuary north of the dam is represented by a northern reservoir (called storage basin in Figure 2-4) and south of the closure dam a separate basin where the water moves up and down with the tide (called sea in Figure 2-4). The closure is represented by the channel between the reservoirs. The northern basin is filled by the tide from the southern basin, with parametrised energy losses caused by the closure dam. The advantage of this model is the short simulation time (~1s).The

model cannot consider the slope in water level in the norther basin away from the dam. As a result, the model overestimates water flowing through the closure during the construction leading to an overestimation of the flow velocity, resulting in overestimating of the required stone size. The effect increases when the length of the storage basin becomes larger.

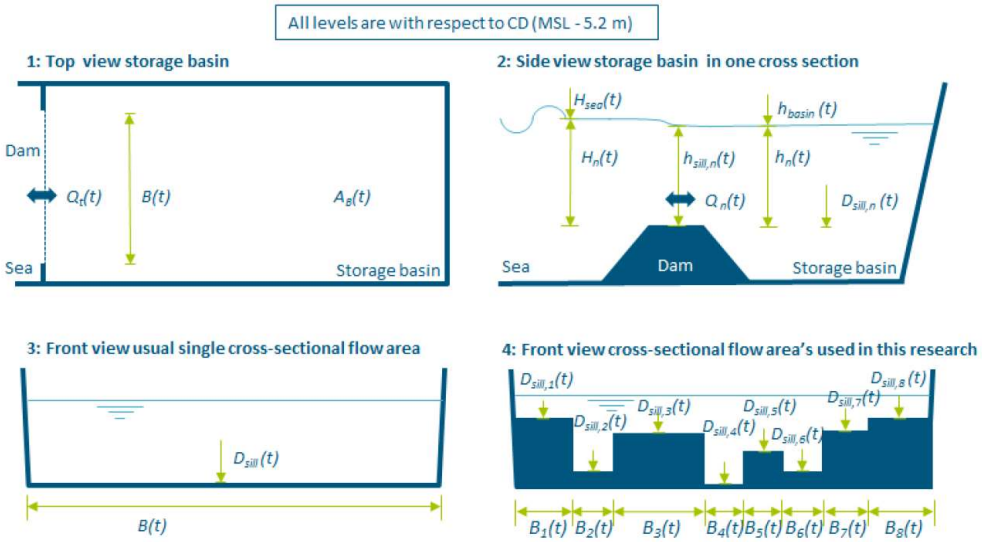


Figure 2-4: The storage model used by [4] to analyse the closure procedure.

1-D model

Another model which is considered is called SOBEK, see Figure 3-1. The SOBEK model is a one-dimensional river model. It has a simulation time of ~20s. It includes, among other things, a spatial variation of the water over the estuary, can be forced by the tidal evaluation south of the closure and includes the storage of water in each segment of the model. The limitation of this model is that the full 2D and 3D of the flow are not included.

2-D model

The third model considered is Delft3D. An example of such a model is depicted in Figure 2-5. It includes 2D variation of the water level over the entire domain, it can also model the wind and density induced currents. Wave can also be included in the model. A drawback of this software is the long computation time, ~48 hours using a dedicated modelling machine.

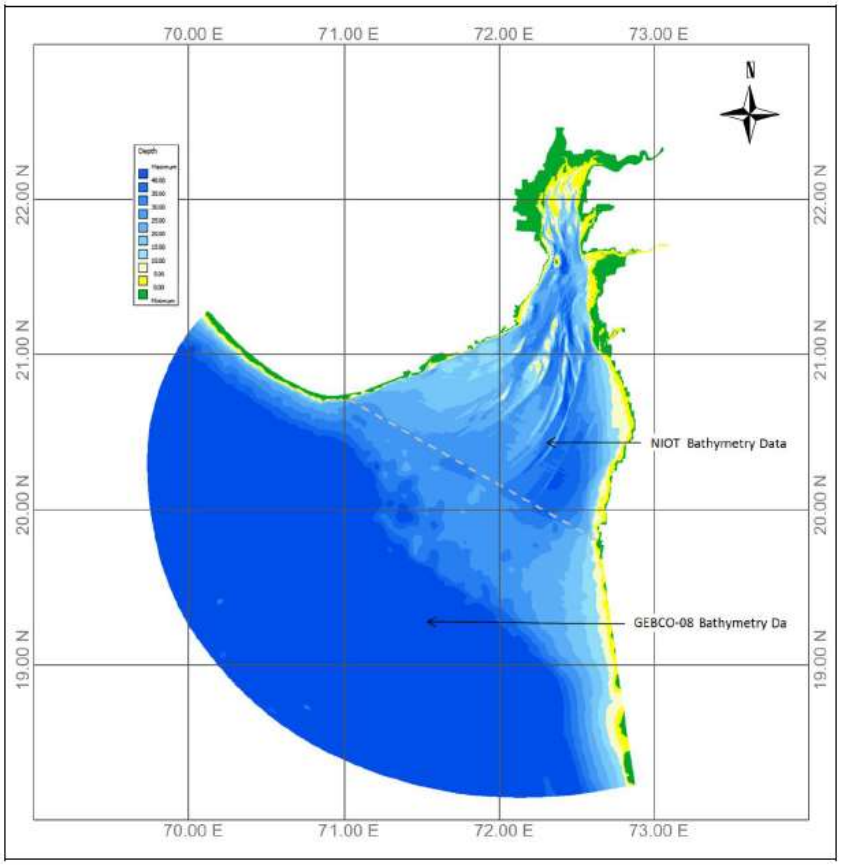


Figure 2-5: The bathymetry of a full Delft3D of the project area. Source: [5].

The above explanation provides insights in the different models and the (dis)advantages. The storage model is considered to be too crude for a realistic representation of reality. On the other hand, the advantage of a more physically realistic spatial representation of water flow in the 2-D model is countered by the limitations of long simulation times, which makes it practically impossible to use in this stage of the project. For these reasons, in this project the SOBEK model will be applied. The simulation time is sufficiently short to simulate hundreds or even thousands of scenarios. This is essential to consider all the different stages and configurations of the closure. Additionally, this model includes the spatial variation of the water level along the estuary resulting in more realistic lower flow velocities compared to the storage model. The model is considered sufficiently accurate for the purpose of this study.

3 Theoretical background

For the closure methodology study, the hydrodynamic simulations will be carried out using the SOBEK 1DFLOW module (version 2.16) developed by Deltares [2]. The SOBEK 1DFLOW module is based on the Saint-Venant equations for unsteady flow with the following assumptions [2]:

- The flow is one-dimensional, i.e. the velocity can be represented by a uniform flow over the cross-section and the water level can be assumed to be horizontal across the section.
- The streamline curvature is small and the vertical accelerations are negligible, hence the pressure is hydrostatic.
- The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady flow.
- The average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replace by unity.

The Saint-Venant equations that are solved in SOBEK are the continuity equation and momentum equation that describe the conservation of mass and momentum in 1D:

Continuity equation:
$$\frac{\partial A_T}{\partial t} + \frac{\partial Q}{\partial x} = q_{lat} \quad (1)$$

Momentum equation:
$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A_F}\right) + gA_F\frac{\partial \zeta}{\partial x} + \frac{gQ|Q|}{C^2RA_F} - w_f\frac{\tau_{wind}}{\rho_w} + gA_F\frac{\xi Q|Q|}{L_x} = q_{lat} \quad (2)$$

with A_T the total area (flow + storage) [m^2], Q the discharge [m^3/s], q_{lat} the lateral discharge per unit length [m^2/s], A_F the flow area [m^2], C the Chezy constant [m$^{1/2}$/s], g the gravitational acceleration [m/s^2], ζ the water level [m], L_x the length of branch segment [m], R the hydraulic radius [m], t the time [s], w_f the water surface width [m], x the distance along channel axis [m], ρ_w the water density [kg/m^3], τ_{wind} the wind shear stress [N/m^2], and ξ an extra resistance coefficient [s^2/m^5].

Together with a set of initial conditions and boundary conditions, the Saint-Venant equations are used to solve the water levels and discharges at specific points. The initial conditions are water levels and/or discharges at the start of the simulation. Similarly, discharges or water levels (constant or time-varying) can be imposed at each boundary node as boundary conditions.

A SOBEK model comprises a network of branches connected to each other at connection nodes. In each branch, calculation points are defined to represent the spatial numerical grid. SOBEK uses a staggered grid for the simulations. This means that the calculation points at which the water levels are computed differ from the locations where the velocities are computed. Water level calculation points (ζ- calculation points) are provided by the user and velocity calculation points (u-calculation points) are placed halfway between the water level calculation points. The continuity equation is solved between the u-calculation points and the momentum equation between the ζ- calculation points. An illustration of a computational grid in SOBEK is given in Figure 3-1.

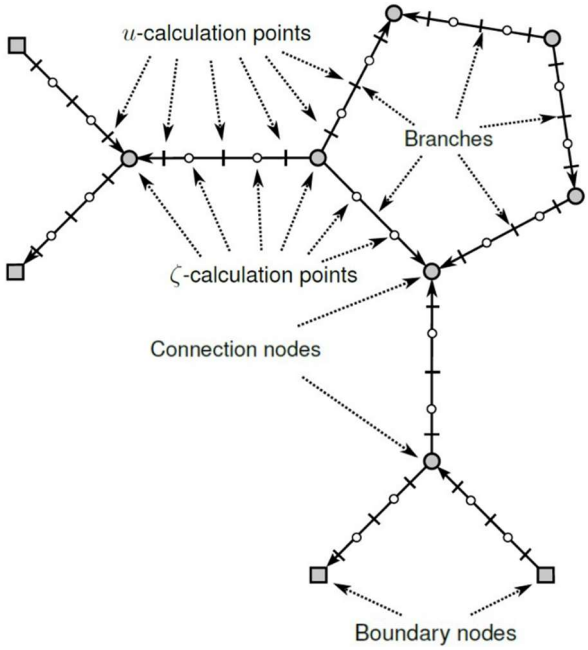


Figure 3-1: Illustration of a staggered grid in SOBEK [10]

4 Model setup

This chapter describes the model setup. It only describes the general model settings and not the case specific settings. The case specific settings are described in Chapter 6: Results. This is done because the case specific settings are strongly dependent on the results of previous simulations.

4.1 Computational grid

To model the flow velocities during the closure operation, a network of branches and nodes was created in SOBEK. Along the reaches, 67 nodes were added where cross-sections of the bathymetry were defined. The bathymetric data from [6] is used. The bathymetric data of these cross-sections are interpolated for the calculation nodes in between the cross-section nodes. Figure 4-2 shows the created network together with the locations of the cross-sections and the underlying bathymetry depicted in Figure 4-2.

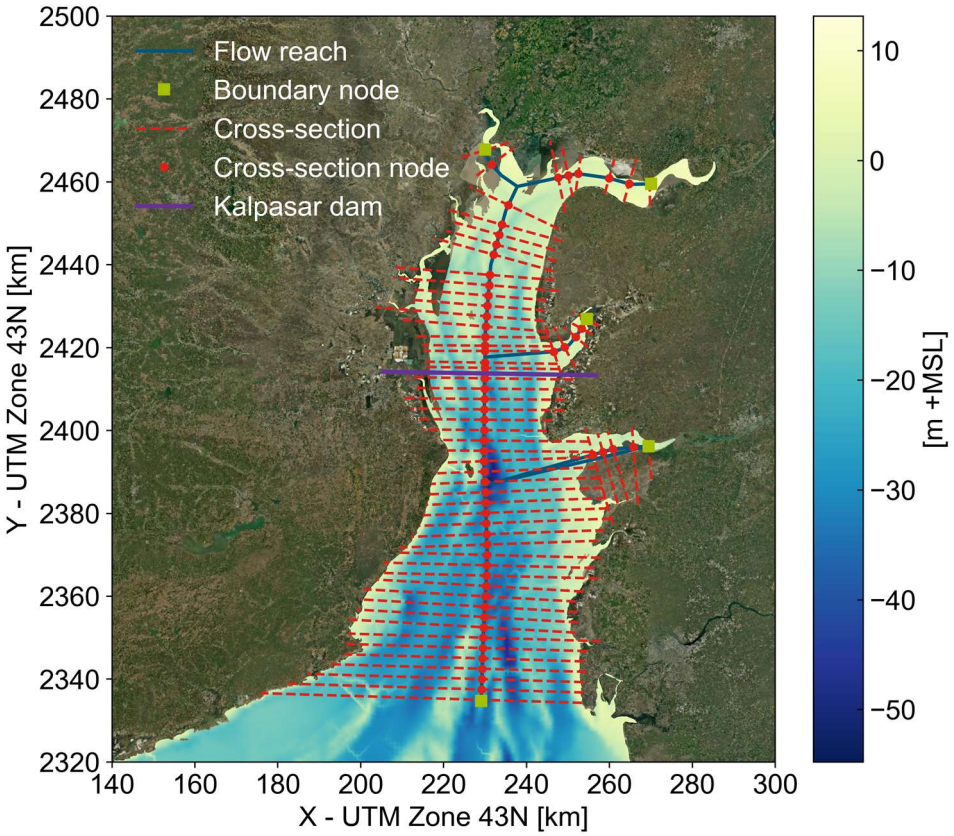


Figure 4-1: The SOBEK model combined with the bathymetric data from [6]. The red dots in the location of the cross-sections and the purple line the location of the Kalpasar dam.

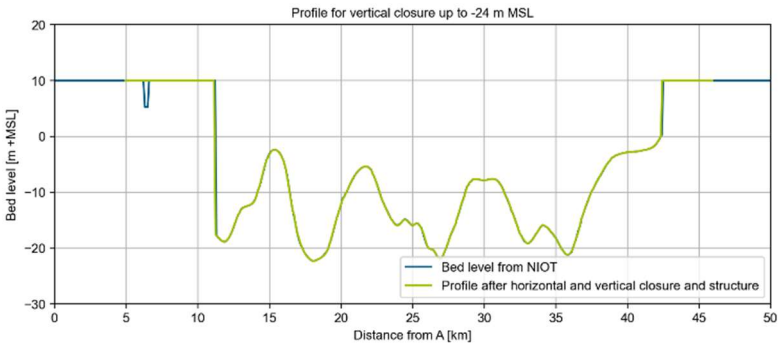


Figure 4-2: Cross-section along the Kalpasar dam.

4.2 Boundary conditions

Boundary conditions are imposed in the model at the ends of every network reach, see Figure 4-3. These boundary conditions can either be a discharge or water level. For the most southern boundary node, a timeseries of a representative spring tide is imposed. This timeseries can be obtained from 30-min interval measurements at several tidal stations for the year 2013. The SOBEK model is a 1D model while there are two tidal stations near the southern boundary of the model (Gopnath and Hazira, see Figure 4-3). This means that a choice has to be made which water level measurements to impose as a boundary condition. There is a substantial difference between the tidal elevations at Gopnath and Hazira. The difference in amplitude can be in the order of 0.5 m (Figure 4-4) with the tidal amplitude at Hazira larger than at Gopnath.

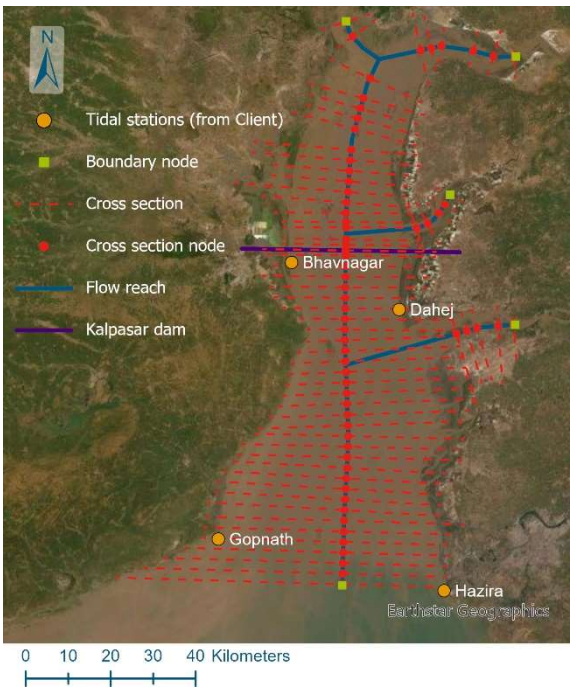


Figure 4-3: Tidal stations within model domain

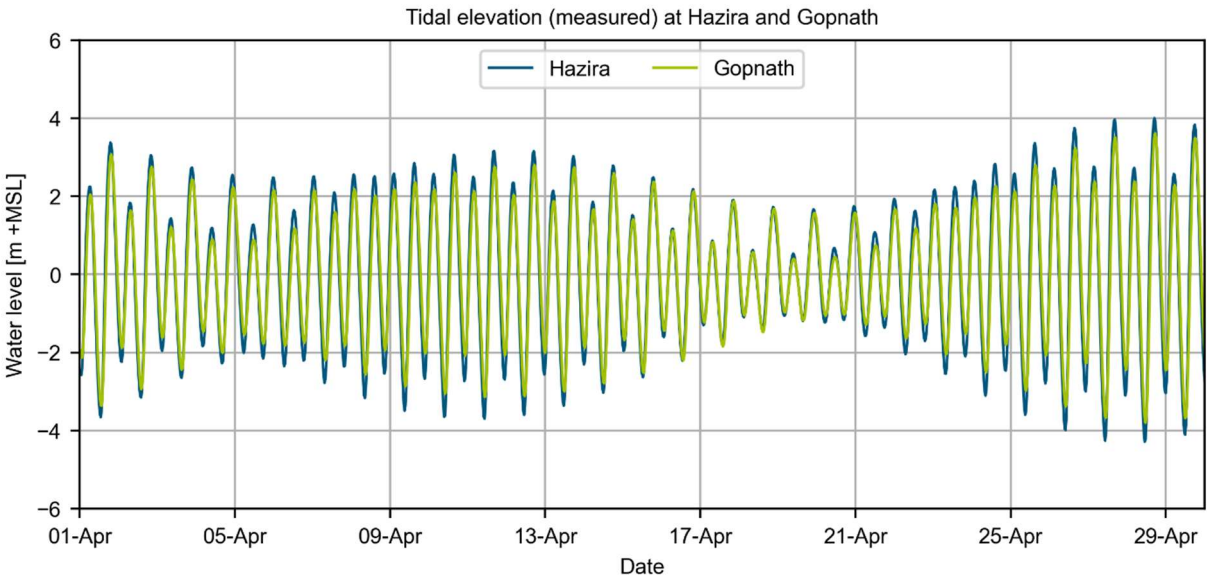


Figure 4-4: Tidal elevations at Hazira and Gopnath for April 2013

Moreover, there is a phase difference between the tidal elevations at both stations (Figure 4-5). This phase difference follows from the large-scale tidal movement around India. The phase difference between the east and west side of the basin disappears as the tide propagates into the Gulf of Khambhat (Figure 4-6). In reality, the phase of the tidal signal in the estuary lays between the phase at Hazira and the phase at Gonpath. Because a single tidal signal needs to be chosen, it is expected that a phase difference arises between the modelled and measured tidal signal in the estuary. However, this is not a major issue because the purpose of this modelling study is to determine the maximum flow velocity and not the moment when the maximum flow velocity occurs.

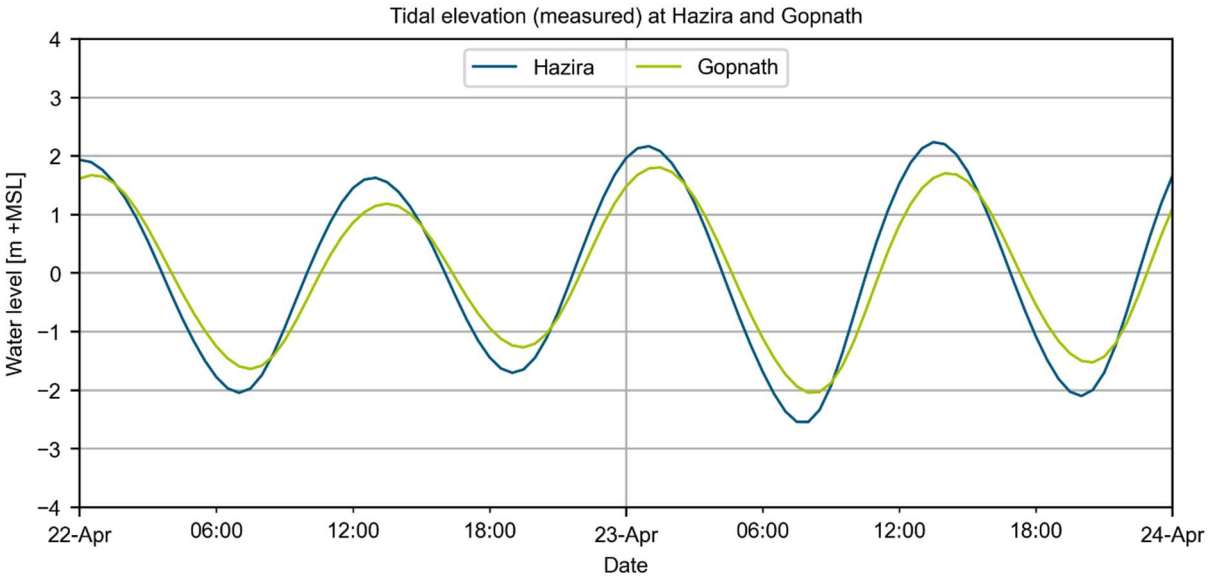


Figure 4-5: Tidal elevations at Hazira and Gopnath for the period 22 to 24 April 2013

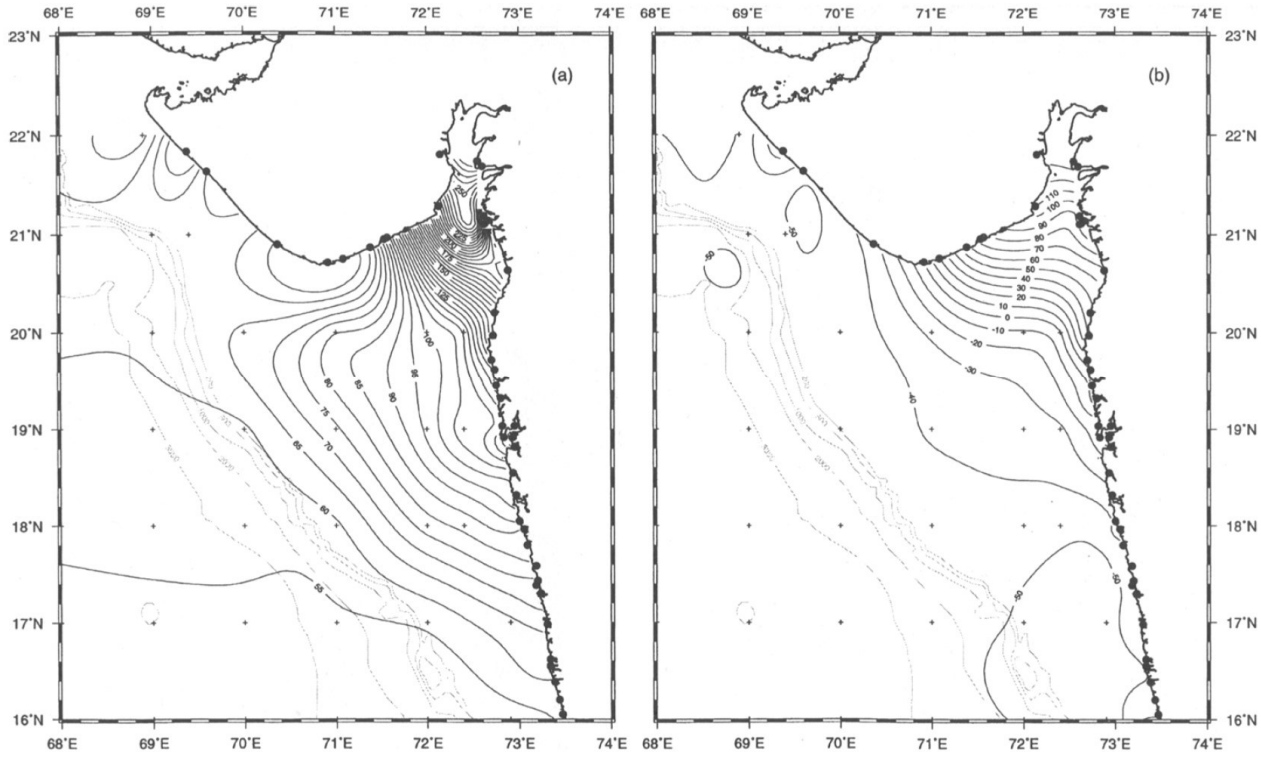


Figure 4-6: (a) Amplitude (cm) and (b) phase (degrees) of the tidal constituent M_2 in the Gulf of Khambhat [7]

For the simulations, the measurements tidal signal at Hazira is used. The modelling results, presented in Chapter 6, reveal that using the tidal signal at Hazira provides better results compared to the measured tidal signals at Bhavnagar and Dahej. Additionally, choosing the tidal signal at Hazira is a conservative choice because the tidal elevation at Hazira is higher than at Gopnath leading to higher flow velocities at the dam location.

At the other boundary nodes, river discharges are imposed. The discharge data was obtained from the "Global River Discharge Database" that contains a compilation of monthly mean discharge data at measurement stations worldwide [8]. The chosen discharges values for the boundary nodes are discussed in more detail in Chapter 5 where the calibration is discussed.

4.3 Hydraulic roughness

The hydraulic roughness or the bed friction value is a crucial input value for the SOBEK simulations. It is the main calibration coefficient in the model. Friction values can be defined for each network branch individually or for the entire network. Several friction formulations can be applied, e.g. Chézy, Manning, Strickler or White-Colebrook. The friction values will be adjusted such that the model results match the available water level measurement in terms of tidal amplitude, shape, and timing. Considering that the friction coefficient is used as a calibration coefficient, the exact value of this parameter cannot be compared with values based on field measurements. On the other hand, the value should remain within physical bounds. In this study, the calibration focusses on the water level measurements near the closure because those measurements are available. It would be better to calibrate on measurements further up north but that data is not available.

4.4 Implementation of Closure methods

In Section 2.1: Closure methods, a horizontal, vertical and sudden closure have been introduced. The SOBEK model needs to consider these methods and needs to be able to model combinations of these settings. A horizontal closure can be modelled by the Sobek model changing the land boundary further seaward, see Figure 4-7. A vertical closure can be modelled by increasing the bed level as shown in Figure 4-8. For the sudden closure **open caissons** are considered which are placed at the start of the closure dam. The placing of the open caissons is extremely challenging and if they would be placed in a later stage of the construction, it is expected that this could not be done because of the extreme hydraulic conditions. The open caissons are represented as weirs in the numerical model. To visualise them , green blocks are plotted in the cross-sections; in reality they are of course open, see Figure 4-9. These closure steps can also be combined in the cross-section of the SOBEK model as shown in Figure 4-10.

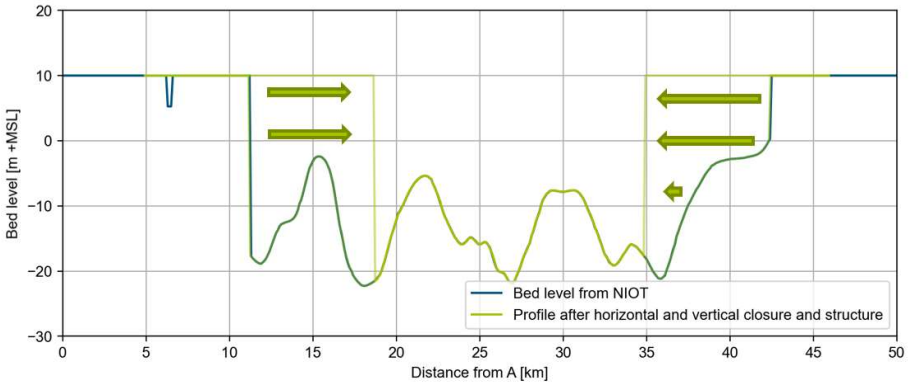


Figure 4-7: A horizontal closure of 15 km. The blue line is the original bed level and the green line the bed level for the closure. Note: Bed levels are extracted from NIOT study [6].

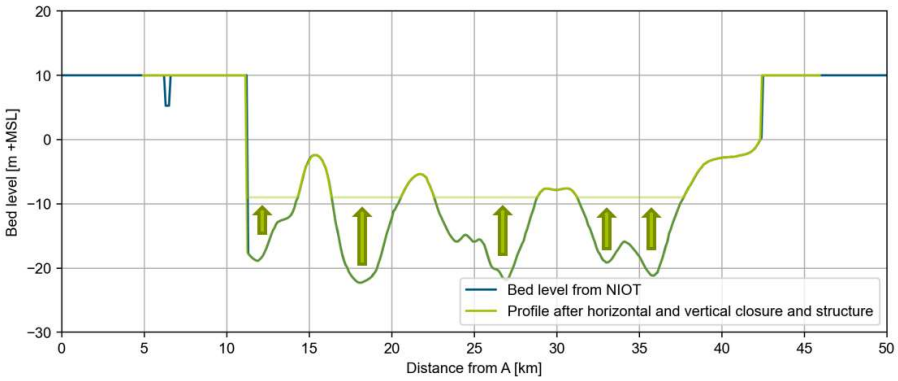


Figure 4-8: A vertical closure of 15 km. The blue line is the original bed level and the green line the bed level for the closure. Note: Bed levels are extracted from NIOT study [6].

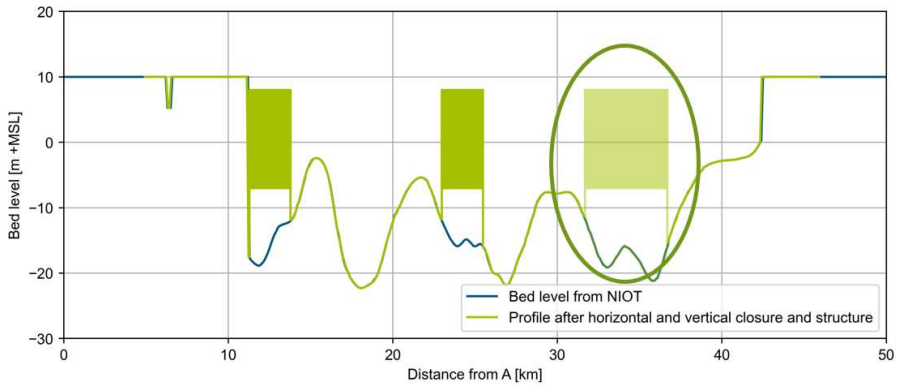


Figure 4-9: A increase in the total length of open structure from 5 km to 10 km. The blue line is the original bed level and the green line the bed level for the closure. Note: Bed levels are extracted from NIOT study [6]. The green blocks show the location of the open caissons.

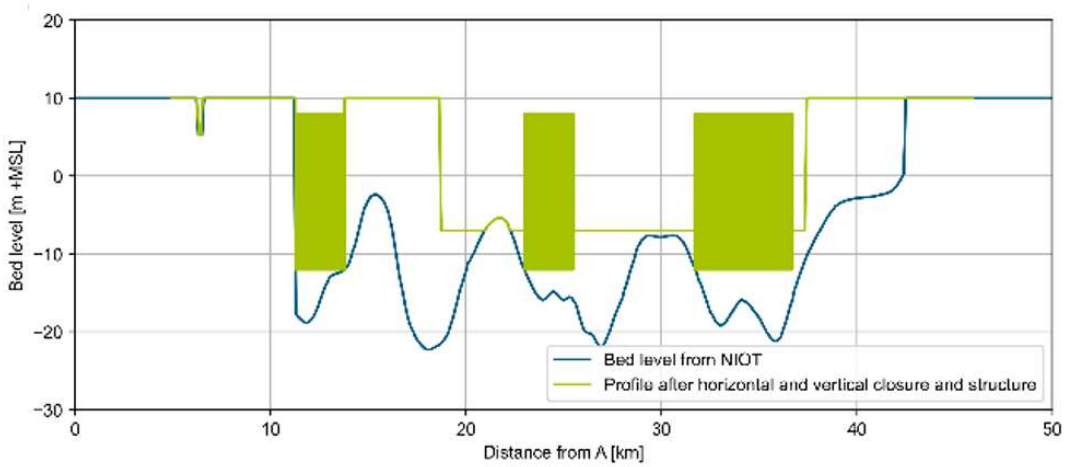


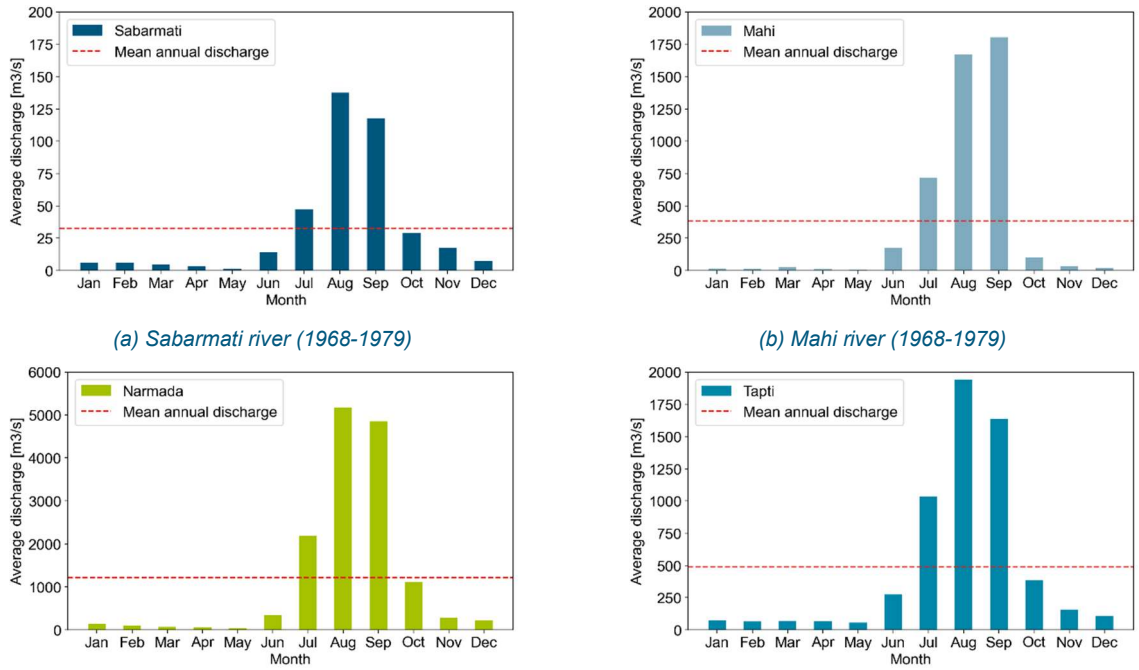
Figure 4-10: A closure dam consisting of 10 km of open caissons (green block), a horizontal closure of 10 km and a vertical closure of 17 m. The blue line is the original bed level and the green line the bed level for the closure. Note: Bed levels are extracted from NIOT study [6].

5 Model calibration

This chapter describes the calibration of the SOBEK model. First, the approach of the calibration process is explained. The subset of the dataset chosen to perform the calibration on, i.e. calibration period, and the calibration parameter are discussed. Then, the calibration results are presented.

5.1 Calibration approach

The variations in water levels in the Gulf of Khambhat are a result of the tide and, to a small extent, the discharges of the rivers flowing into the Gulf. Ideally, the calibration procedure should cover both aspects. However, timeseries of the river discharges matching the timeseries of the tidal stations are not available. Because the effect of the tides is dominant over the river discharge, the calibration focusses on reproducing the tidal levels in the estuary. To further minimize the error made by the discharge of the 4 rivers April 2013 is selected as model period. During this period the monthly averaged river discharge is very low, see Figure 5-1.



(a) Sabarmati river (1968-1979)

(b) Mahi river (1968-1979)

(c) Narmada river (1949-1979)

(d) Tapti river (1940-1979)

Figure 5-1: Mean discharges of the major rivers per month (data period)

The water level measurements of the tidal stations, shown in Figure 4-5 were used for the calibration of the SOBEK model. The calibration was performed by adjusting the friction formulation and corresponding friction coefficient in SOBEK such that the simulated water levels correspond sufficiently well to the measurements at the tidal stations. The friction formulations and coefficients that were considered in the calibration exercise are listed in Table 5-1. Two formulations are considered Manning and de Bos & Bijkerk. The friction formulation by Manning is suitable for a range of hydraulic conditions which are extensively covered in literature, see for example [9]. The Bos & Bijkerk friction formulation was included because this formulation incorporates the water depth next to a friction parameter into its formula for the flow resistance. The water depth is thus taken into account more explicitly, which might be fitting given the large water depths in the Gulf of Khambhat. The values of the coefficients presented in Table 5-1 are based on what is deemed physically realistic.

Table 5-1: Bed friction conditions considered for the calibration of the SOBEK model

Sim no.	Friction formulation	Value	Unit	Notation
1	De Bos & Bijkerk	20	1/s	BB_20
2	De Bos & Bijkerk	25	1/s	BB_25
3	De Bos & Bijkerk	30	1/s	BB_30
4	De Bos & Bijkerk	35	1/s	BB_35
5	Manning	0.015	$s/m^{1/3}$	MA_015
6	Manning	0.020	$s/m^{1/3}$	MA_020
7	Manning	0.025	$s/m^{1/3}$	MA_025
8	Manning	0.030	$s/m^{1/3}$	MA_030
9	Manning	0.035	$s/m^{1/3}$	MA_035
10	Manning	0.040	$s/m^{1/3}$	MA_040

5.2 Calibration results

A typical timeseries of the modelled and measured surface elevation is depicted in Figure 5-2 and Figure 5-3. The figures reveal a clear similarity between the modelled and measured surface elevation. A small difference in phase is clearly visible but this difference will have a neglectable effect on the modelled results because the goal is to determine the maximum flow velocity rather the moment in time this maximum velocity occurs.

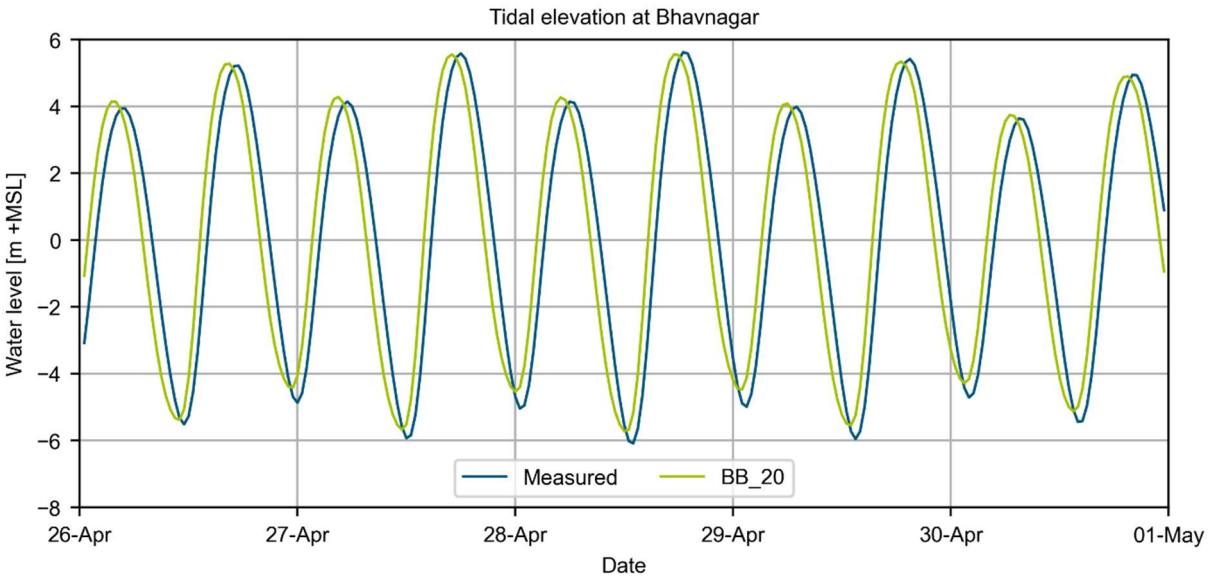


Figure 5-2: The modelled and measured water levels at Bhavnagar for 26 April to 1 May 2013 for Simulation 1, see Table 5-1.

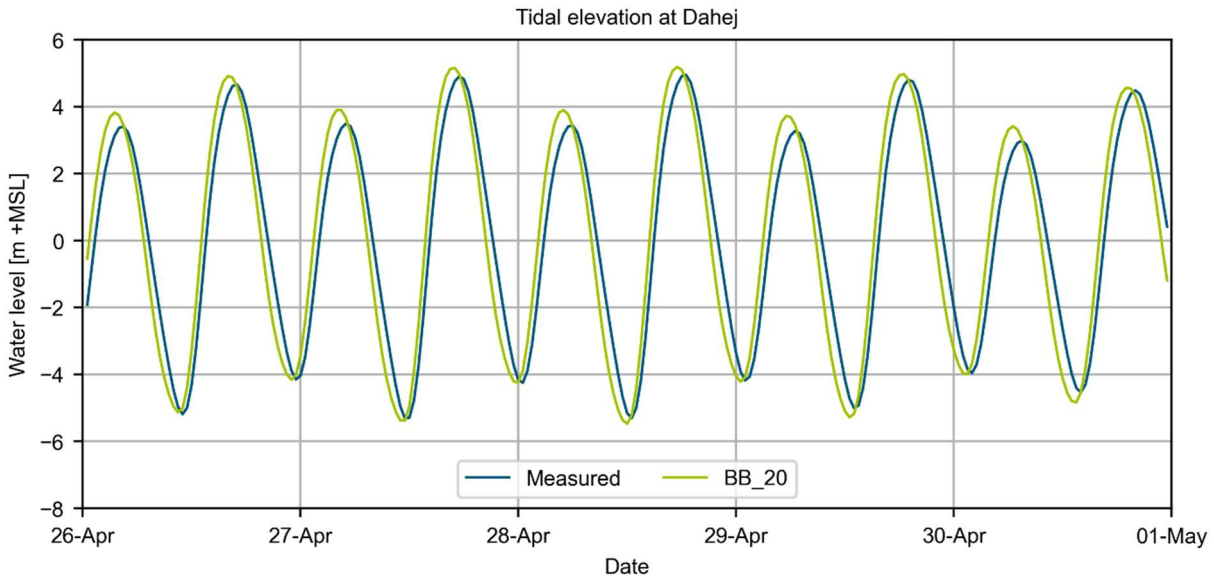
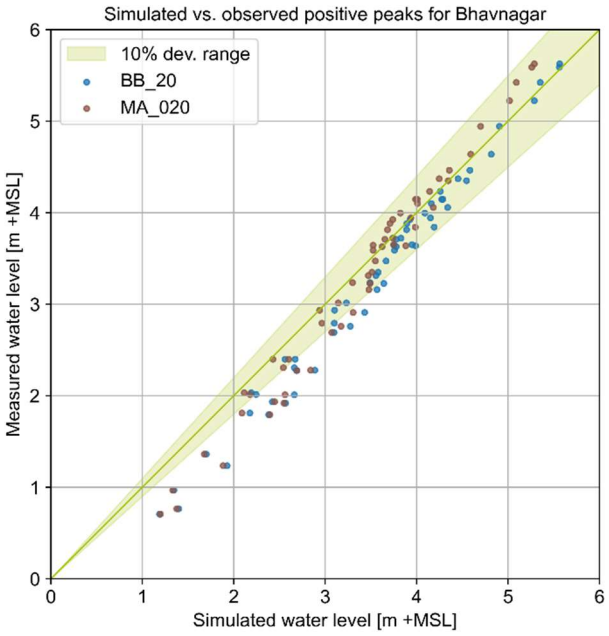


Figure 5-3: The modelled and measured water levels at Dahej for 26 April to 1 May 2013 for Simulation 1, see Table 5-1.

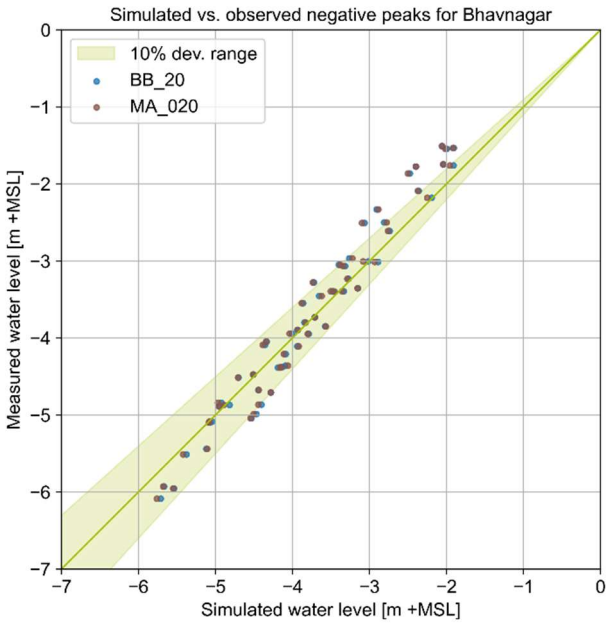
The surface elevation for each simulation of Table 5-1 is determined. From the time signals the individual maxima and minima are determined and compared with the measured maxima and minima. The best results are found for the Bos & Bijkerk friction relation the best results are found for $\gamma = 20$ 1/s (BB_20) and for Manning with $n_m = 0.020$ s/m$^{1/3}$ (MA_020). Scatter plots of those results are presented inFigure 5-4 and Figure 5-5. The figures show little difference between the two friction conditions in terms of deviations from the line of perfect agreement. Especially for Dahej, the simulations seem to correspond well to the measurements.

As the amplitudes are slightly more often underestimated for Manning with $n_m = 0.020$ s/m$^{1/3}$ (MA_020) than the Bos & Bijkerk formulation with a $\gamma = 20$ 1/s (BB_20), the friction formulation by de Bos & Bijkerk with coefficient $\gamma = 20$ 1/s (BB_20) is used for future simulations. The difference between the measured and modelled results are considered small enough for this stage of the project.

Although the model is calibrated, and the water level is reasonably well represented at the location of the closure dam, there are many limitations of the model. It is still a 1D model and therefore the variation of flow velocity along the 30km dam alignment is ignored. With a considerable variation of the depth along this line, the model gives only an estimate of the average velocity which locally can be very different. Also, the likely morphological effects of the closure (erosion and sedimentation) are neglected while these can be considerable. In light of the purpose of this model which is a tool for a quick comparison of many scenarios this is considered acceptable and sufficient to determine a technical feasible closure method.

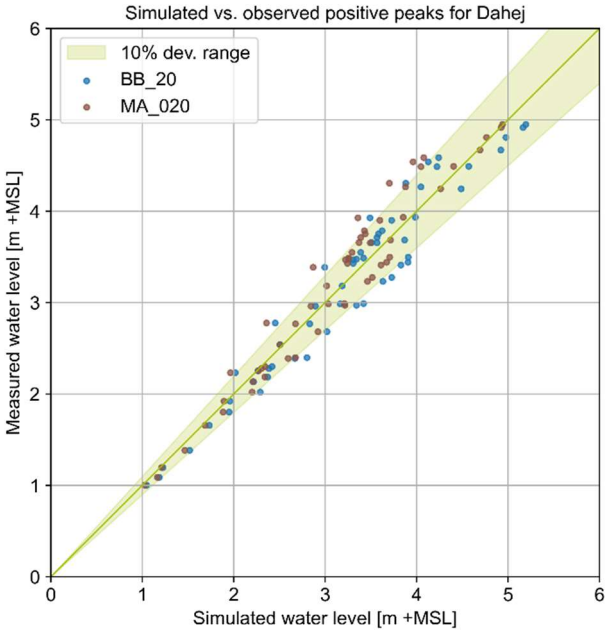


(a) Comparison of positive peaks at Bhavnagar

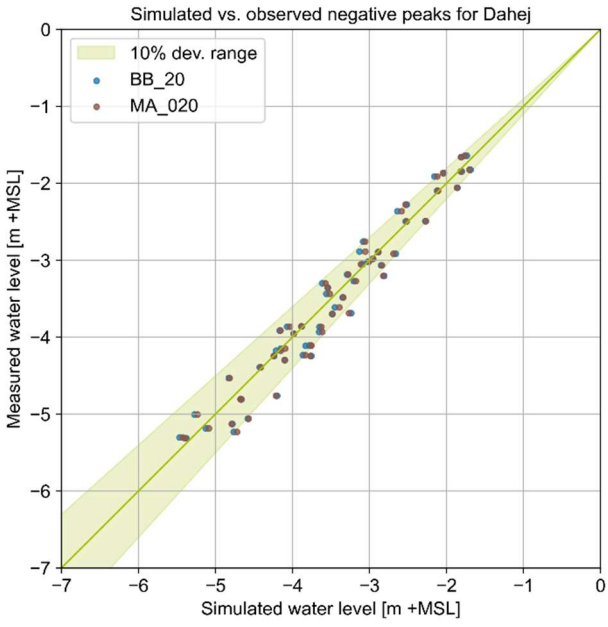


(b) Comparison of negative peaks at Bhavnagar

Figure 5-4: Comparison between simulated and measured peaks at Bhavnagar for BB_20 and MA_020



(a) Comparison of positive peaks at Dahej



(b) Comparison of negative peaks at Dahej

Figure 5-5: Comparison between simulated and measured peaks at Dahej for BB_20 and MA_020

6 Results

In this chapter, the results of the modelling efforts are presented. The chapter is divided into three sections corresponding to the three stages proposed for the modelling work. First, in Section 6.1, the results of the Exploratory stage (Step 1) are discussed. Section 6.2 describes the results of Step 2, the Production stage. The results of the optimization stage will be presented in a separate addendum report.

6.1 Step 1: Exploratory stage

Step 1 was introduced to get a better understanding in the importance of different design criteria, for example different caisson properties and river discharges. Hence, specific simulations were selected to provide this understanding. In total 31 simulations have been performed in this stage. The different simulations have specific settings for the horizontal and vertical closure. In other words, the simulation demonstrates a specific phase of the closure rather than the entire closure. In Step 2 full closure sequences will be modelled.

6.1.1 Simulations without caissons

As a starting point of the investigations, the basic cases for the full horizontal and full vertical closure have been assessed.
The results for these type of closure are summarized in the figure below. The figure shows that for a full horizontal closure the maximum velocity reaches as high as 10 m/s and for a full vertical closure the maximum flow velocity is limited to 5.9 m/s. These flow conditions are considered too high and therefore a closure without any caissons is considered not feasible.

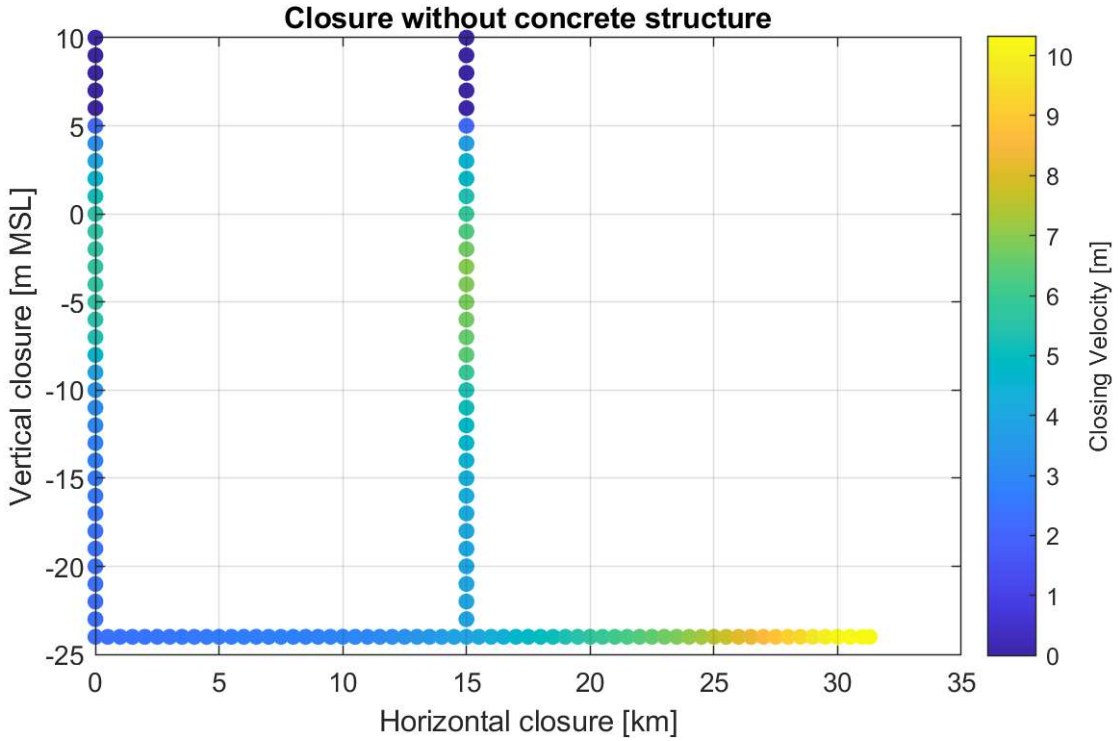


Figure 6-1: The maximum flow velocity for a full horizontal and full vertical closure. A 50% horizontal closure followed by a 50% vertical closure is also shown.

6.1.2 Considered variants

Multiple properties with multiple design variants are considered in Step 1. An overview of these properties and their considered values are given in Table 6-1. The definitions are also presented graphically in Figure 6-2. In this study we are only considering open caissons. Each caisson contains multiple gate openings. "Total length of the open structure ($\sum_{i=1}^{n} L_i$)" is the sum of the entire length of the open gates in all caissons. This variable is equal to the number of caissons times number of gates in each caisson time the size (width) of each gate opening. Each opening in a caisson has a height H and a width (W). The thickness of the walls and floors of each caisson is neglected in this study. Therefore, the height of the caisson is equal to the height of the gate and equal to the height of the opening. The width of each caisson is equal to the number of gates in each caisson times the width of each gate. Note, the wall friction is included in the model.

Table 6-1: Considered variables and their investigate variations. The bold values are their default values.

Variable	Variations
Caisson variables	
Total length of the open structure ($\sum_{i=1}^{n} L_i$)	2.5, 5, 7.5, **10**, 12.5 km
Number of sections (n)	1, 2, 3
Caisson height (H)	15, **20** m
Gate width (w)	10, **25**, 50 m
Environmental variables	
River discharge	**None**, medium, large
Vertical closure depth	-10, **-8**, -6 m MSL
Horizontal closure distance	**0**, 5 km

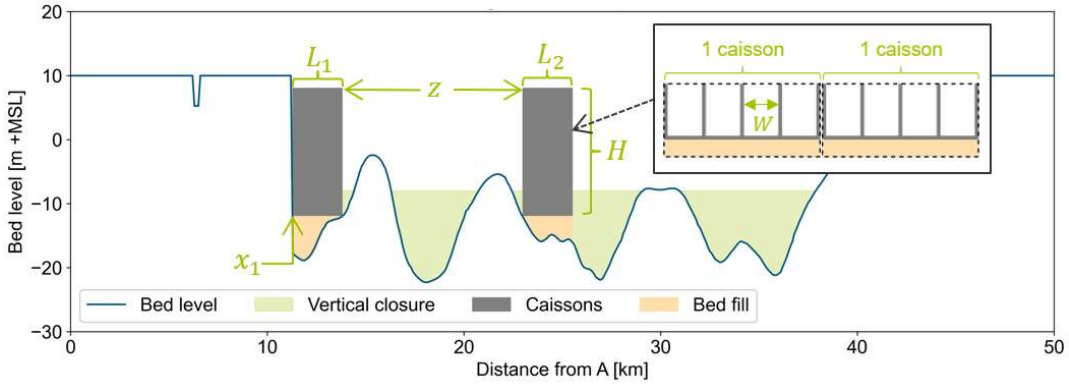


Figure 6-2: Definitions of variables related to the caisson configuration that are varied in the exploratory stage.

Table 6-1 includes the number of sections. Since the total length of the open structure is in the order of kilometers and the caisson width is in the order of 50 m, the total number of caissons is in the order of hundreds and therefore placing them will be time consuming. By dividing the total structure into multiple sections, multiple placing fronts are created, and this might give a shorter total placing time. Therefore, combining a number of sections is also investigated. There is no default value selected because the results will show that the effect of the number of sections is neglectable.

The vertical closure depths in Table 6-1 were chosen such that the effects of the caisson configurations are noticeable in the velocity profiles of the simulations, but not too large to ensure that the flow remains subcritical and the flow conditions over the dam do not obscure the impacts of caisson configurations or river discharges. Selecting a specific vertical level means that only a specific moment of the vertical closure is investigated. In Step 2 full horizontal and vertical closing strategies are modelled.

In the variations with medium river discharges, simulations were run including river discharges corresponding to the mean discharges of the month August shown in Figure 5-1. In the simulations with large river discharges, the probable maximum flood values mentioned in the spillway capacity study by the Indian Institute of Technology Roorkee [10] and the pre-feasibility study by Haskoning from 1998 [11] were used. The values are summarised in Table 6-2. For the simulations with extreme river discharges (Large 1 and Large 2), two conditions are included; with and without the contribution of the Narmada river. These simulations are performed as a sensitivity analysis the investigate the effect of the river discharge. These scenarios have been selected as purely hypothetic to examine the extremes, they are not meant to represent reality, e.g. no flow in the Narmada river is not realistic, it has been added to test the sensitivity of the system to a very high river discharge from the north and no discharge from the south (the Narmada discharge increases the water level in the estuary south of the closure dam, reducing the head difference across the closure dam).. The effect of a maximum river discharge for the Sabarmati, Mahi and Dhadar river combined with a maximum discharge for the Narmada river might be different from the scenario when only the Sabartmati, Mahi and Dhadar river have their maximum discharge.

Table 6-2: River discharge conditions considered in the exploratory stage, no river discharge is the default value.

Condition	Sabarmati [m^3/s]	Mahi [m^3/s]	Dhadar [m^3/s]	Narmada [m^3/s]
Medium	120	1,800	-	4,860
Large 1	37,000	33,300	8,400	120,000
Large 2	37,000	33,300	8,400	-

In Chapter 2 different closure techniques were discussed: horizontal closure, vertical closure and sudden closure. In the numerical model the horizontal closure will be modelled by moving the land boundary towards the centre of the cross-section. A vertical closure will be modelled by raising the bed level. The sudden closure open caissons are considered. During the closure the water can flow through them the entire time. When the horizontal and vertical closure is finished, all the gates in the caissons are closed at once. In step one specific moments of a closure consisting of a horizontal, vertical and sudden closure are modelled. In step 2 full horizontal and vertical closing strategies are modelled for different configurations of the open structure.

6.1.3 Modelled flow velocity

A typical geometry with the modelled flow velocity is presented in Figure 6-3. Compared to the reference situation an increase in flow velocity due to the closure is clearly visible. For each simulation similar figures are made. To make the simulations easier to compare we will only be investigating the maximum flow velocity. In this case the maximum flow velocity is 3.5 m/s. The maximum flow velocity without any closure is 2.3 m/s. An overview of all the simulations of Step 1 and their maximum flow velocity is given in Appendix A.

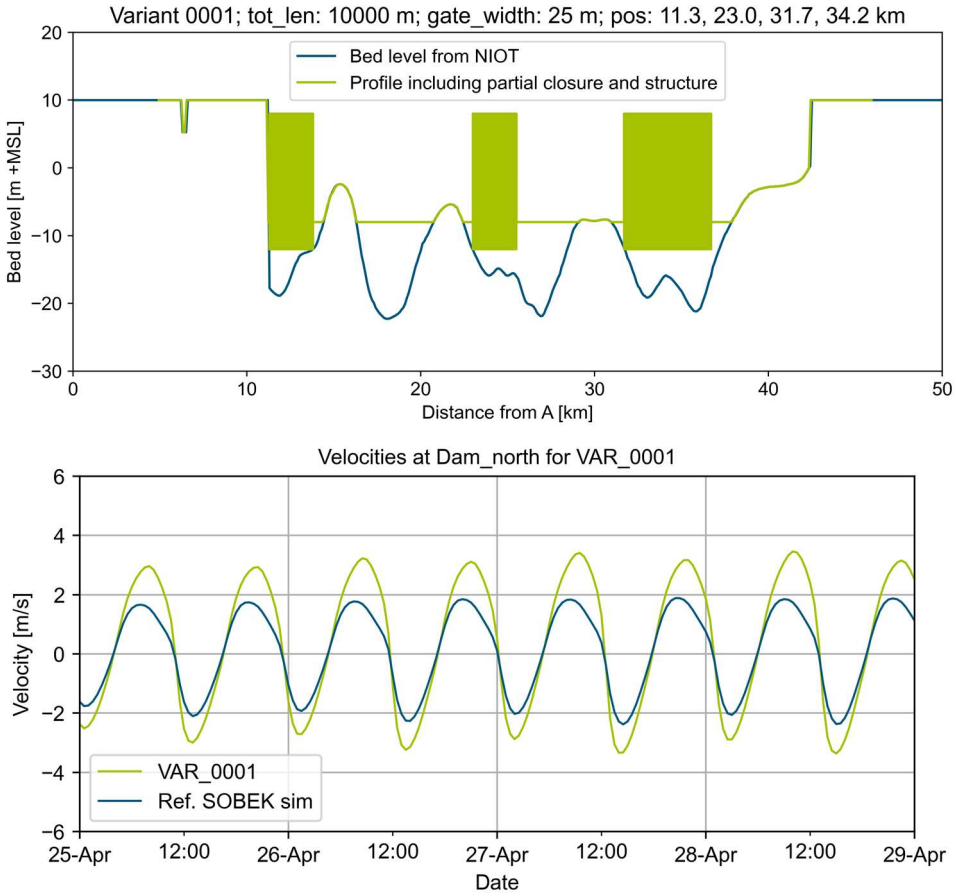


Figure 6-3: The geometry of the default cross-section (top) with the blue line the bathymetry before any closure is build, the green blocks are 3 sections of concrete structure and the green line the new bed level during the closure. Note: Bed levels are extracted from NIOT study [6]. The bottom figure presents the velocity in this cross-section. The blue line in the bottom figure presents the flow velocity for the fully open situation.

To investigate the impact of the number of sections the geometry shown in Figure 6-3 is adjusted to the geometry presented in Figure 6-4. The number of sections in reduced from 3 to 2. The maximum flow velocity stays the same at 3.5 m/s, as expected. By reducing the number of sections, the number of sides by the caissons is changed. This additional friction does not affect the maximum flow velocity significantly. This is because the sides between the gates, see Section 6.1.2 , are also included in the model. The effect of a single additional side is too small compared to the total number of open sides already in the model. Therefore, the reduction of 2 section sides is neglectable.

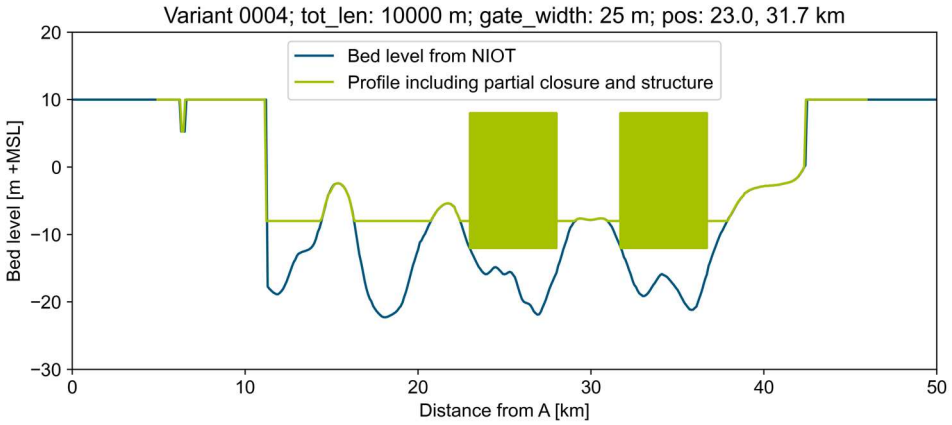


Figure 6-4: The geometry with 2 sections rather than 3 to investigate the effect of the number of sections. Note: Bed levels are extracted from NIOT study [6].

The total length of the open structure can be an important design property. Several simulations have been performed with different lengths of the open structures and different vertical levels. The maximum velocities are summarized in Figure 6-5. The figure reveals, as expected, a strong dependence of the total length of the open structure on the maximum flow velocity. Evidently, the impact of the total length is larger when the level of the vertical closure is higher because the difference in open area becomes larger. The figure also reveals a linear relation between the total open structure length and the maximum flow velocity. This property can be useful in Step 2 to reduce the number of closure scenarios which need to be fully modelled.

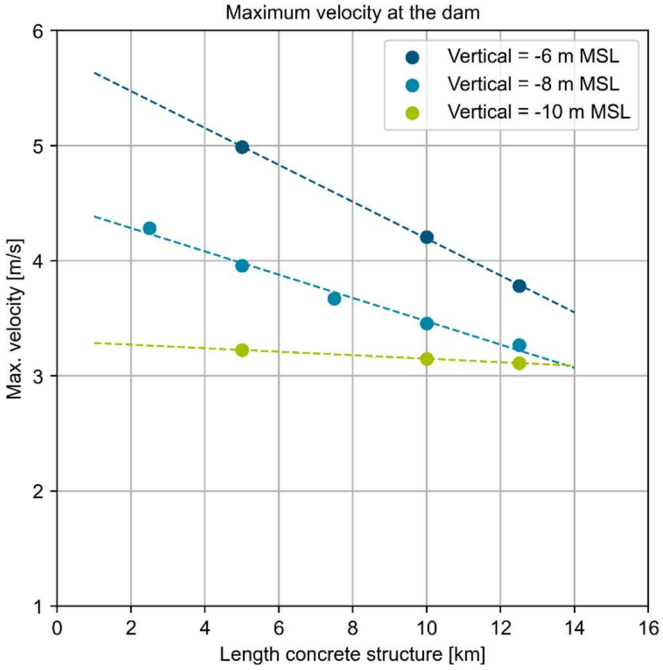


Figure 6-5: Maximum flow velocity at the dam as a function of total length of the open structure for three different vertical closure depths

Another design parameter is the width of the gate. This parameter becomes important because the effect of the walls of each opening is included in the numerical model. The numerical results used to analyse this property are depicted in Figure 6-6. The figure reveals that a smaller gate width increases the maximum flow velocity. It increases with the open structure length as there will be more individual caissons and

thereby more concrete walls blocking the flow. However, the difference between a gate width of 10 m and 50 m is still only about 0.2 m/s for a total open structure length of 12.5 km. In view of constructability and reliability during the closure operation, an optimum should be found between gate width and total number of openings. From a reliability point of view, it is preferred to have as little openings as possible to reduce the probability of failure to close the gate during a sudden closure. At the same time, the maximum gate width is restricted to what is feasible in terms of construction. Based on expert judgement, 25 m appears to be a reasonable value for the gate width.

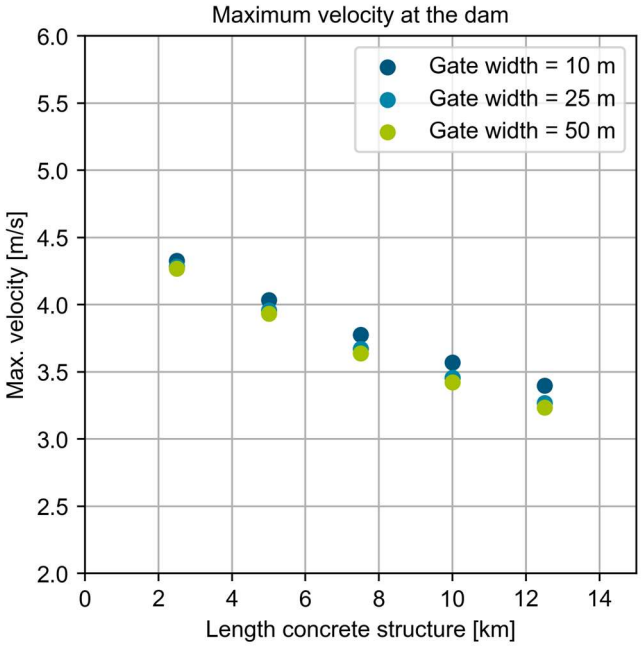


Figure 6-6: Maximum flow velocity at the dam as a function of total length of the open structure for three different vertical closure depths used to analyse the effect of the gate width.

The height of the caissons can also be changed. The top level of the caissons is kept at +8 m MSL, this is because a construction road over the caissons is essential during construction. A reduction of caisson height means that a larger part underneath the caisson needs to be filled, see Figure 6-7. The effect of the height of the caisson is clearly visible in Figure 6-8. Decreasing the height of the caissons strongly increases the flow velocity for the same total length of the open structure. This is because with decreasing height of the caissons, the flow area that is blocked increases. Hence, the caissons should be as high as possible to achieve the largest flow area and thereby reduce the maximum flow velocities during closure. From a constructability point of view, a height of 20 m is deemed to be the maximum feasible height.

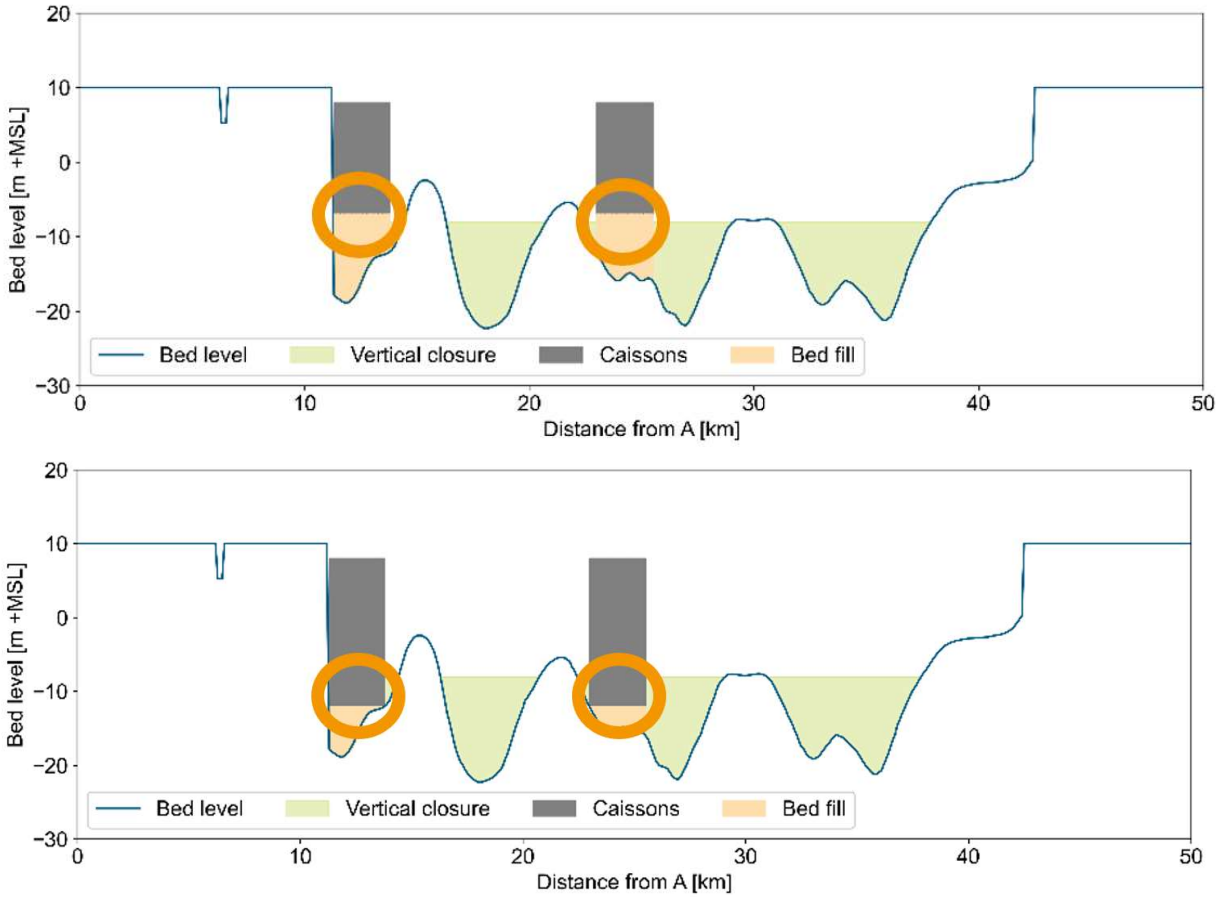


Figure 6-7: The geometries used to investigate the effect of the caisson height. The vertical levels of the caissons are kept equal so the bed fill increases when the height of caissons becomes smaller. The top figure shows a caisson of 15 m and a bed fill till – 7 m MSL The bottom figure depicts caissons with a height of 20 m and a bed fill till -12 m MSL.

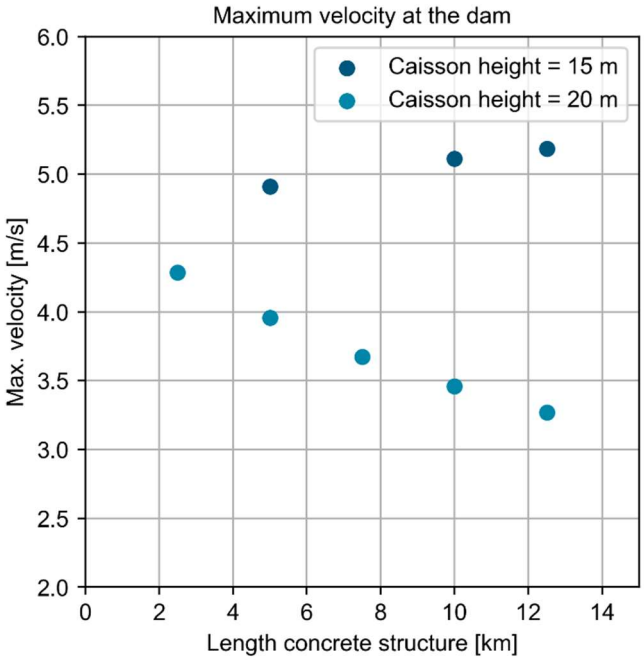


Figure 6-8: Maximum flow velocity at the dam as a function of total length of the open structure for three different vertical closure depths used to analyse the effect of the height of the caisson.

Figure 6-8 shows that increasing the total length of the open structure also increases the flow velocity for a caisson height of 15 m. This is contradictory to the previous finding. This behavior arises because we are modelling with a vertical level of -8 m MSL. For a caisson with a height of 15 m the bed fill is until -7 m MSL. As a result, increasing the total length of the open structure also further reduces overall closure of cross-section. In reality a full closure will need to be performed and this effect would disappear.

In the previous simulations the river discharge has been neglected but it will affect the maximum current velocity. Three different river scenarios have been investigated, see Table 6-2. The results of the different scenarios are presented in Figure 6-9. The influence of river discharges is modest compared to other variables. The figure also shows that the current velocity at the closure dam is dominate by the tidal current and to by the river discharge. There is an increase in maximum flow velocities when extreme river discharges are included, but this increase is fairly small given the magnitude (and probability) of river discharge that was included in those simulations. Moreover, it can be questioned whether it is practical to include these extreme discharges in the simulations for the closure operation. It would be more rational to include some river discharge, e.g. the medium condition, and accept some damage to the temporary structure if the extreme discharges would occur during the construction period.

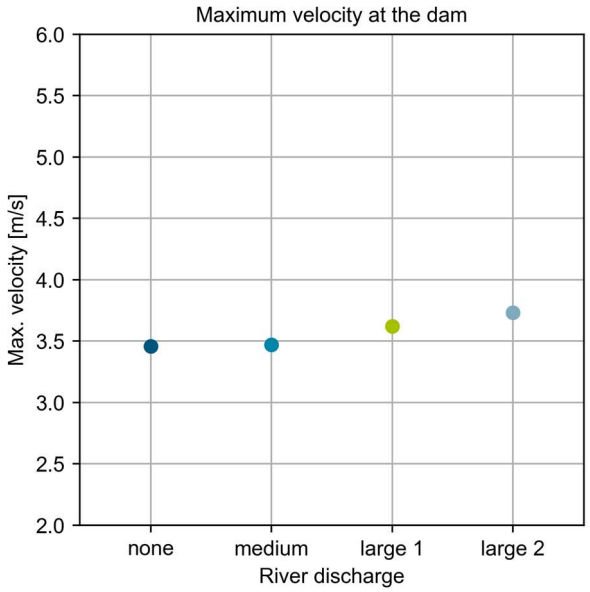


Figure 6-9: The maximum flow velocity for different river discharge scenarios.

To summarize the findings of Step 1:

- The effect of the number of sections on the maximum flow velocity is minimum and can be neglected.
- The maximum flow velocity is strongly dependent on the length of the open concrete structure. When the total length increases the maximum flow velocity reduces.
- The maximum flow velocity is linearly dependent on the length of the open concrete structure.
- The maximum flow velocity increases when the width of the gates is reduced. However, this influence is limited.
- The maximum flow velocity decreases linearly with an increase of caisson height.
- River discharge increases the flow velocity, but the effect is limited.

6.2 Step 2: Production stage

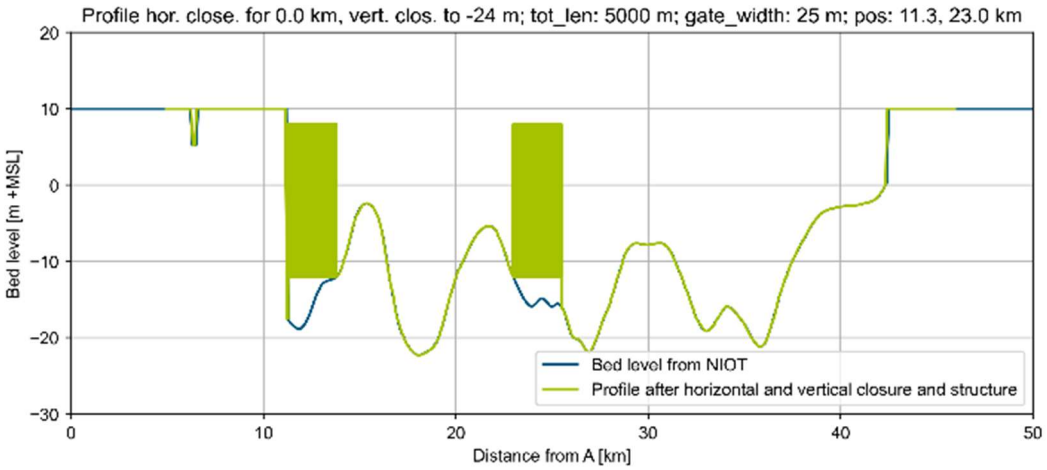
In 1 Step: Exploratory stage the importance of different design criteria has been investigated. Based on that multiple caisson configuration will be selected in Section 6.2.1. For those configurations full closure sequences will be modelled. The number of simulations will be in the order of ~10.000 because the number of combinations of horizontal and vertical closure is large. In Section 6.2.2 the results will be presented of all those combinations.

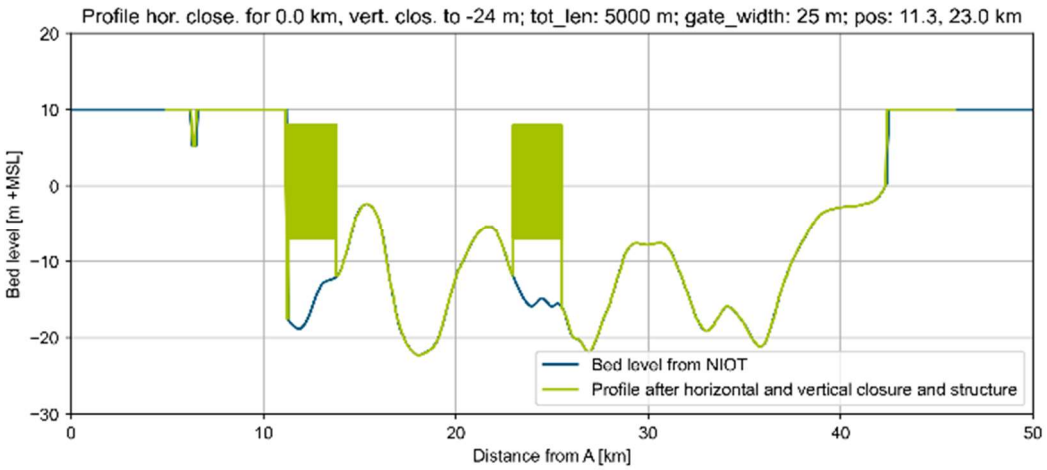
6.2.1 Considered variants

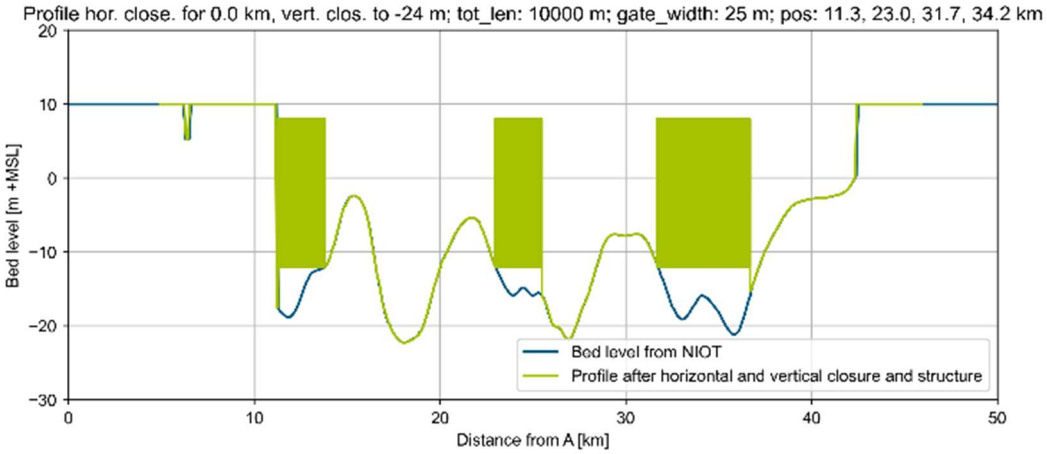
From Step 1 follows that the total length of the open concrete structure and the height of the closure gates are the most important parameters. The total length of the open structure will be set to 5, 10 or 15 km. The step size of 5 km in the total length is chosen to provide a brought spectrum of scenarios. Because the maximum flow velocity is closely linearly dependent on the total length of the open structure, linear interpolation can be applied when the maximum velocity from other structure lengths are needed.

For the caisson height, a height of 15 m and 20 m are considered. Both heights are constructable but are also already considered extremely heigh. It results in 6 different geometries. In Figure 6-10 the 6 geometries are presented and their properties are summarized in Table 6-3. The green line in the figure is

described as the "*Profile after horizontal and vertical closure and structure*". For each closure step a new green line is produced after horizontal







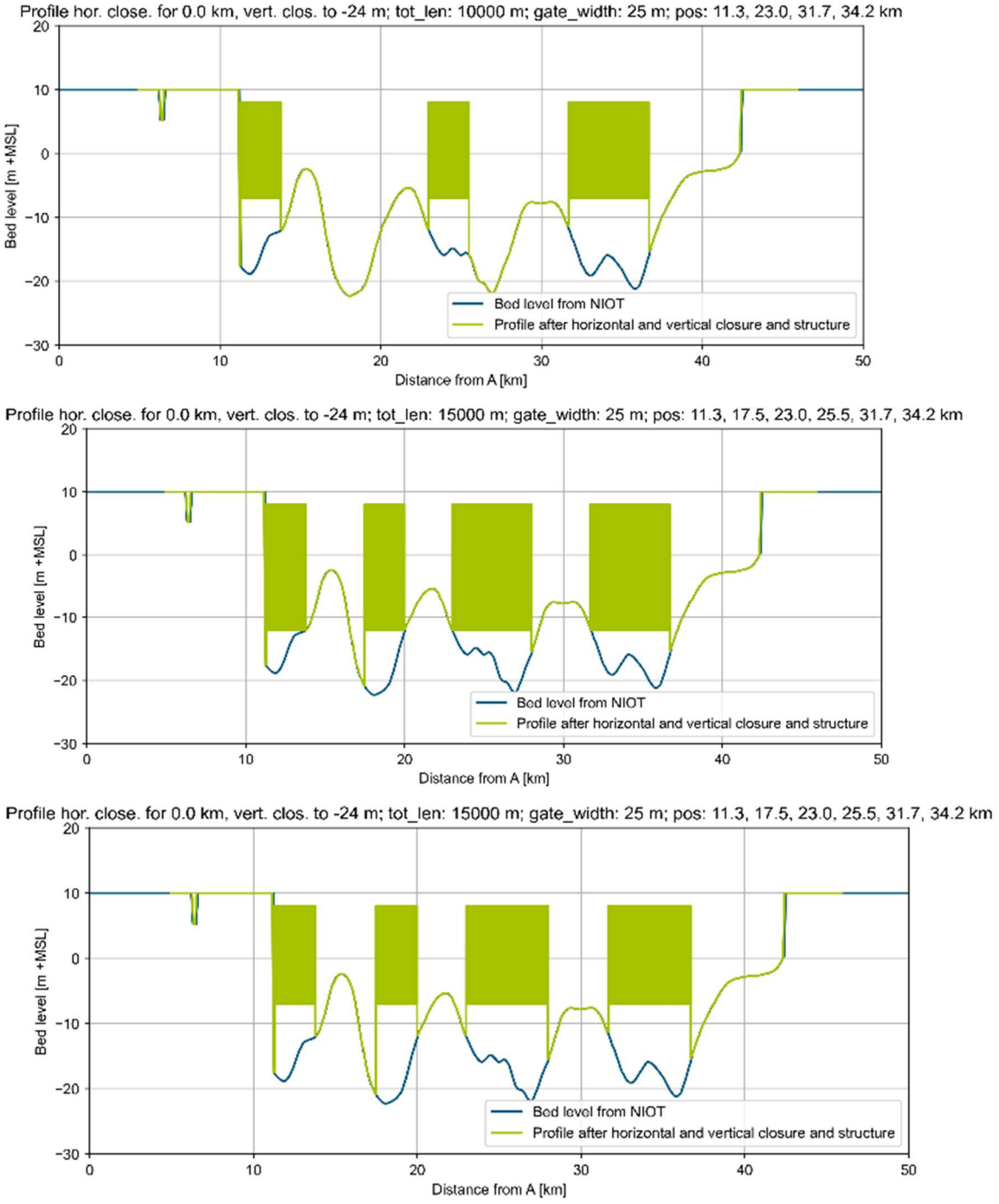


Figure 6-10: The six cross-sections investigated in Step 2. The top two figures have a caisson length of 5 km, the middle two figures: 10 km and the bottom two: 15 km. Cross-section 1, 3, and 5 have a caisson height of 20 m and cross-section 2, 4 and 6 have a height of 15 m, see Table 6-3. Note: Bed levels are extracted from NIOT study [6].

Table 6-3: The different cross-sections considered in Step 2.

Name	Total caisson length [km]	Number of sections	Caisson height [m]
Concrete structure 1	5	2	20
Concrete structure 2	5	2	15
Concrete structure 3	10	3	20

Concrete structure 4	10	3	15
Concrete structure 5	15	3	20
Concrete structure 6	15	3	15

Step 1 showed that the width of the closure gate also influenced the maximum velocity. This property is not investigated further in Step 2 and set to a constant value of 25 m. The influence of the gate width was much smaller than the effect of the height of the caissons or the length of the concrete structure. Also, the width of the gates is also strongly dependent on the constructability. Furthermore, the width of the gates directly affects the number of gates. The number of gates has an enormous effect on the risks during the sudden closure. In other words, the gate width is not considered as a variable because it is more likely determined by the constructability and the acceptable risk during sudden closure than by the maximum flow velocity.

The four rivers boundary conditions are set to a medium scenario. The main reason for applying a river discharge at the boundary is to increase the physical correctness of the model rather than the effect on the maximum flow velocity. Step 1 revealed that the maximum flow velocity is only limited effected by the river discharge.

Analysing ~10.000 simulations is challenging and much of the data is filtered. A schematic overview of the filtering technique is depicted in Figure 6-11. Based on a cross-section of Table 6-3, a length of the horizontal closure and a length of the vertical closure, a cross-section at the closure dam is defined. This cross-section is used in the SOBEK model, and the model is run for a full tidal cycle, see Figure 4-4. It produces a time signal of the flow velocity. The maximum flow velocity is selected and will be used in the analysis of the closing sequence and the design of Volume 2: Technical Report Sudden Closure.

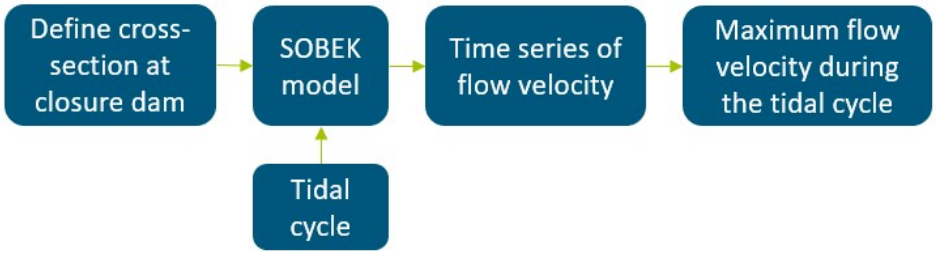


Figure 6-11: A schematic overview of the filtering technique

6.2.2 Modelled flow velocity

An example of the schematic filtering technique is presented in Figure 6-12. The entire SOBEK simulation is summarized by a single point representing the maximum velocity for certain amount of horizontal and vertical closure. For each of the 6 concrete structures a scatter plot is generated. An example of such a scatter plot is given in Figure 6-13. Each dot represents a specific cross-section for which a full tidal cycle is simulated. By combining/connection multiple dots a closure path or closing strategy can be determined.

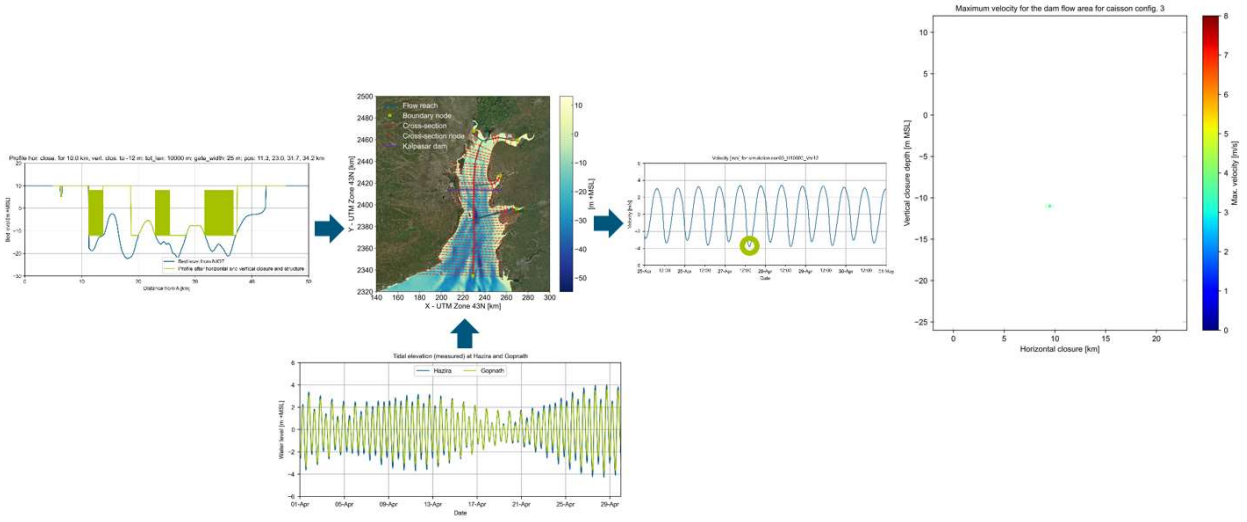


Figure 6-12: An example of the filtering technique based on Figure 6-11 for Concrete structure 3, a horizontal closure of 10 km and a vertical closure till -12 m MSL. Each cross-section provides a single dot in the right figure.

An example of a closing strategy is depicted in Figure 6-13. The blue line indicates the following closure sequence:

1. Placing the caissons
2. Perform a vertical closure till – 19 m MSL
3. Perform a horizontal closure of 5 km
4. Perform a horizontal closure of 2.5 km
5. Perform a vertical closure till -5 m MSL
6. Fill up the rest of dam till 10 m MSL

It is important to realise that the point in the lower left corner of each scatter plot (horizontal closure = 0 and a vertical closure of 0 m) corresponds to a cross-section where the open structure has already been placed.

Figure 6-13 also reveals that the current velocity is continuously increasing for a horizontal closure. For a vertical closure the current velocity is first increasing but after a certain level decreasing. This is because for a vertical closure the flow goes from supercritical to sub-critical.

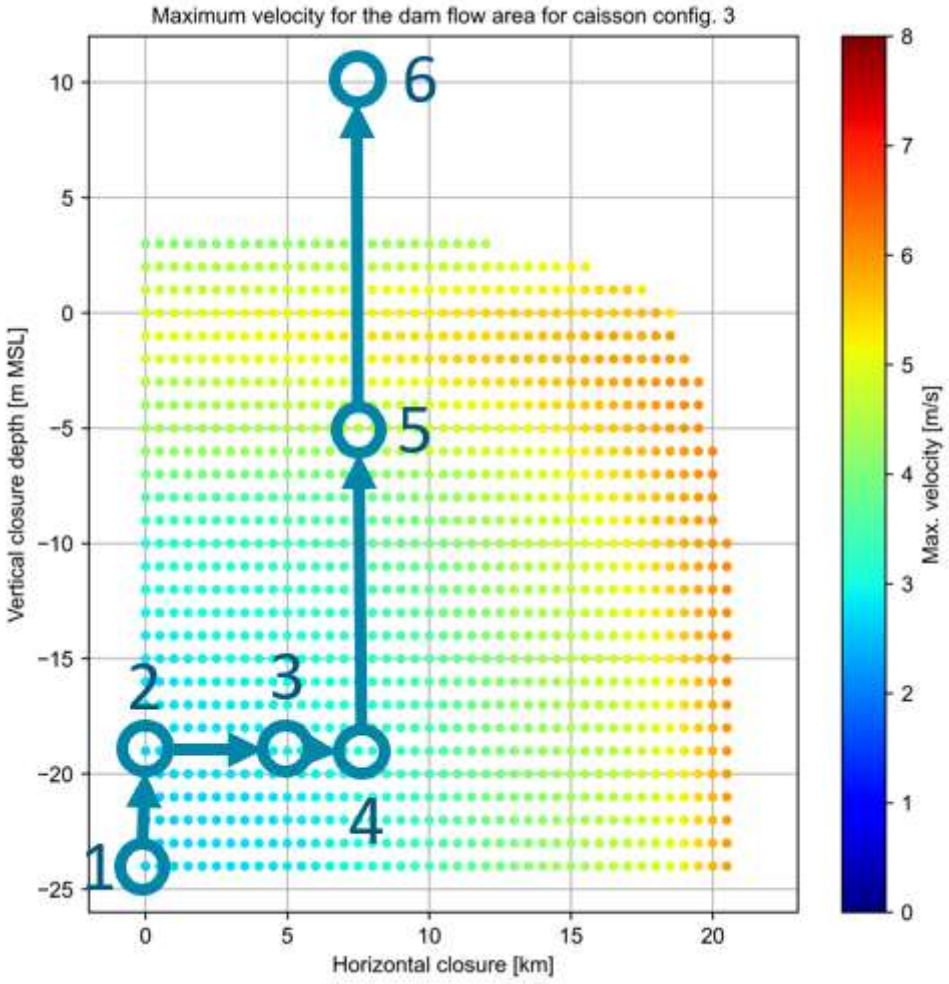


Figure 6-13: An example scatter plot for Concrete structure 3. The blue line is an example of a closure sequence.

The scatter plots for all 6 concrete structures are shown in Figure 6-14 till Figure 6-19. Each path through the scatter plot to the right or top side of the figure represents a closing strategy. It is therefore a very effective way of considering many closure paths and to determine their technical feasibility.

Although many strategies exist to build the closure dam, only a limited number of them are technically feasible. When the maximum flow velocity is higher than 4 m/s the concrete structure will start to erode. This will take a significant amount of time but the construction will also take a considerable amount of time. The caissons will be placed at the beginning of the construction sequence and therefore water will be flowing through them for multiple years.

When the maximum flow velocity increases the required stone size also increases. In the work of [12] a upper limit of 6 m/s is used for the flow velocity. When the maximum flow velocity becomes higher than this value the required rock sizes become challenging. The size of the rocks which are still stable at these flow conditions expensive, difficult to transport and only a limited amount of those rock is available. It is often more attractive to use open caissons in that case. The 6 m/s is a rule of thumb and a required stone size should be determined. This is done in Volume 2: Technical report sudden closure.

When the criterion of 4 m/s is used, 15 km of open structure consisting of caissons with a height of 20 m required for the closure. When the maximum flow velocity is allowed to become 6 m/s 10 km of open

structure consisting of 20 m high caissons or 15 km of open structure consisting of 15 m high caissons are required. The exact amount will be based on the construction time, the construction method and the available rock sizes.

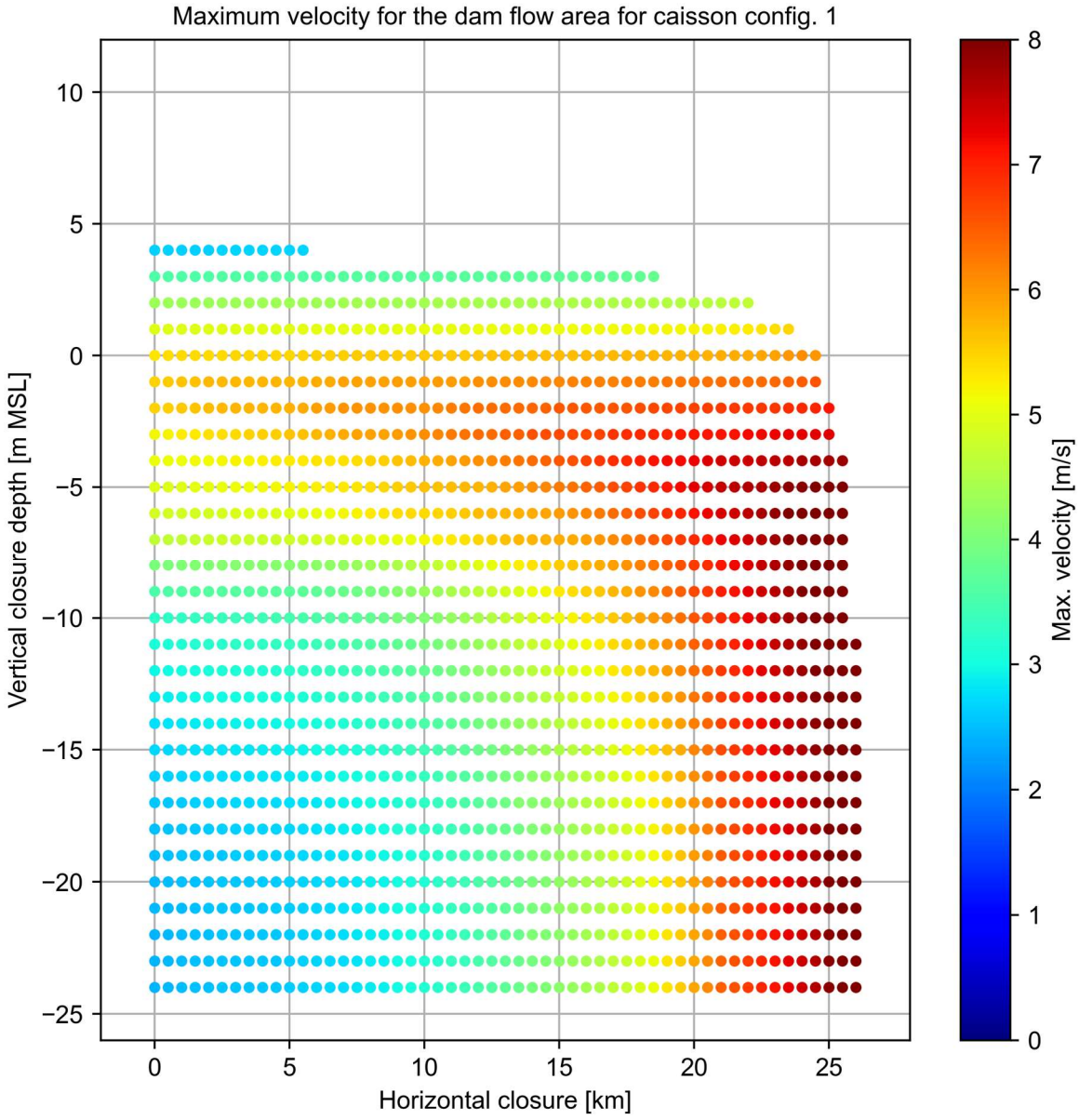


Figure 6-14:The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 1.

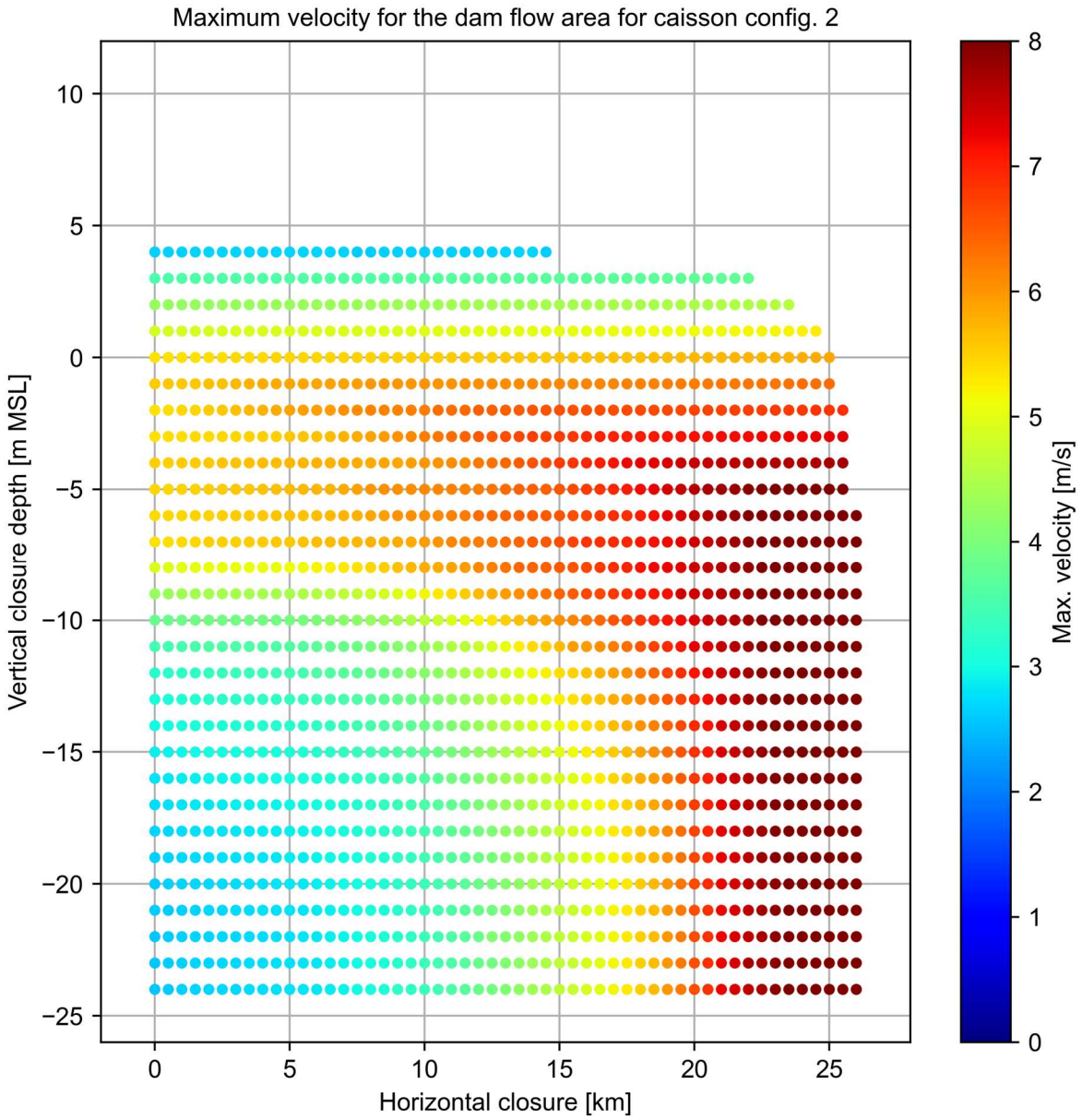


Figure 6-15:The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 2.

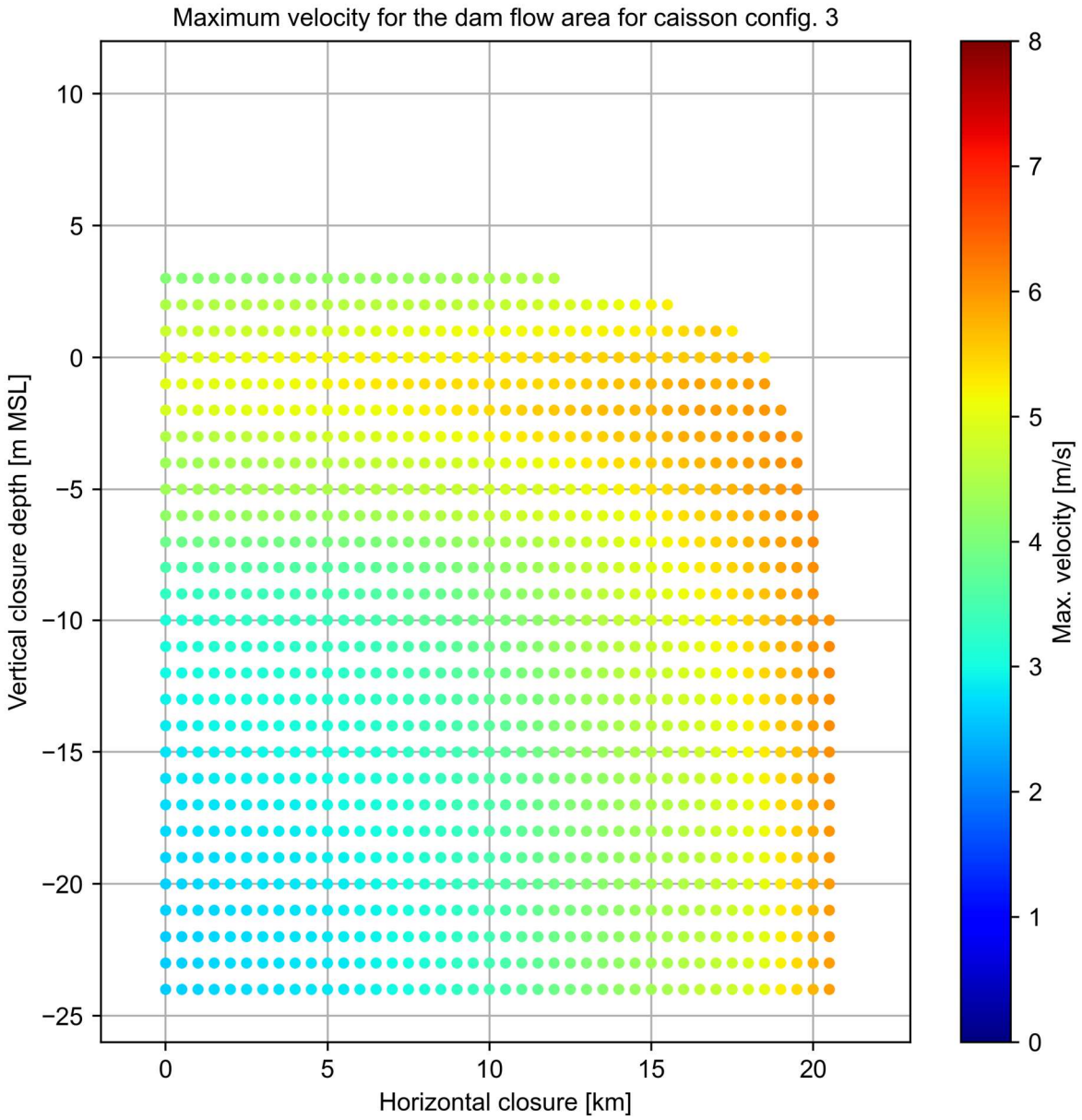


Figure 6-16: The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 3.

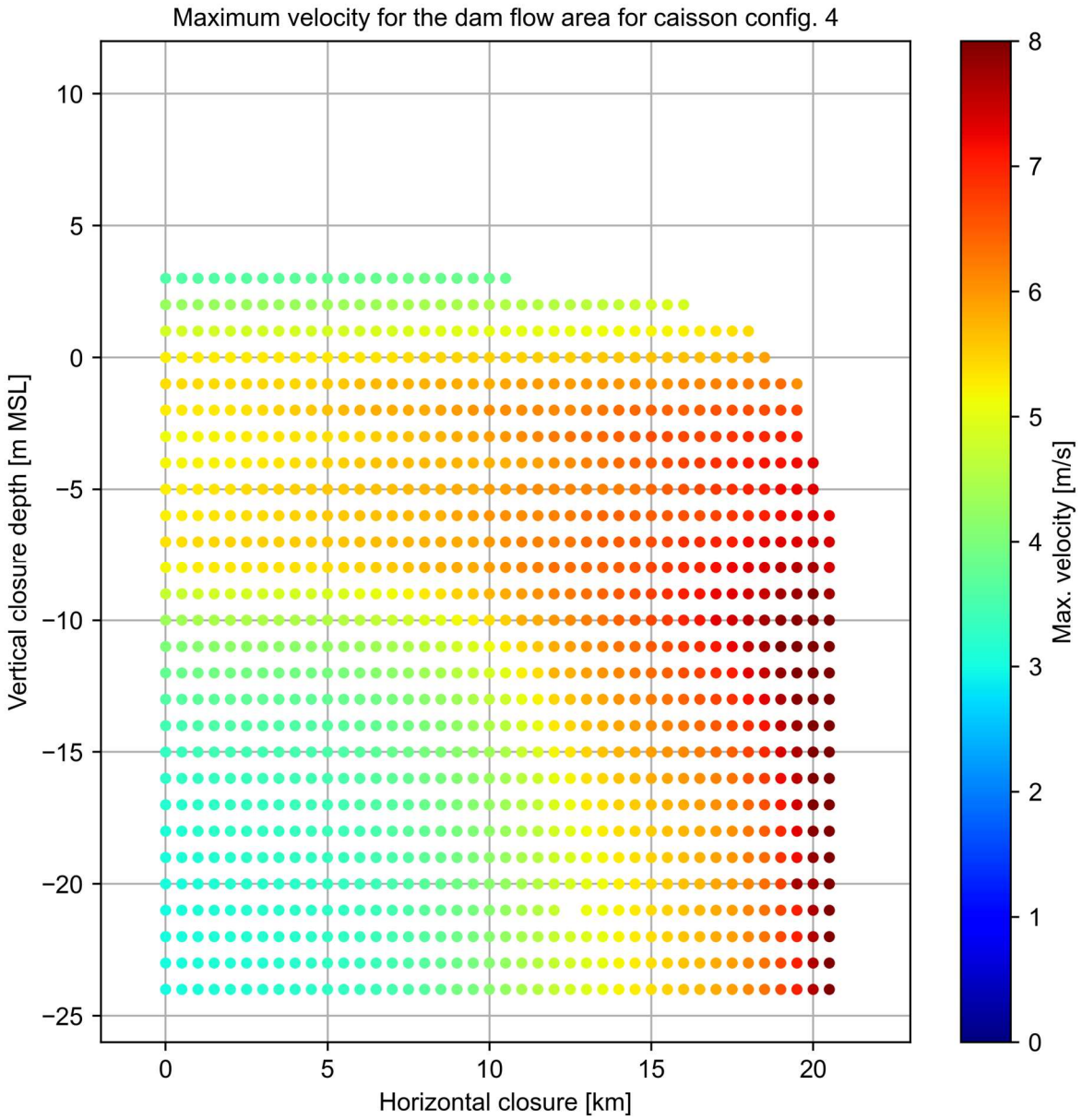


Figure 6-17: The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 4.

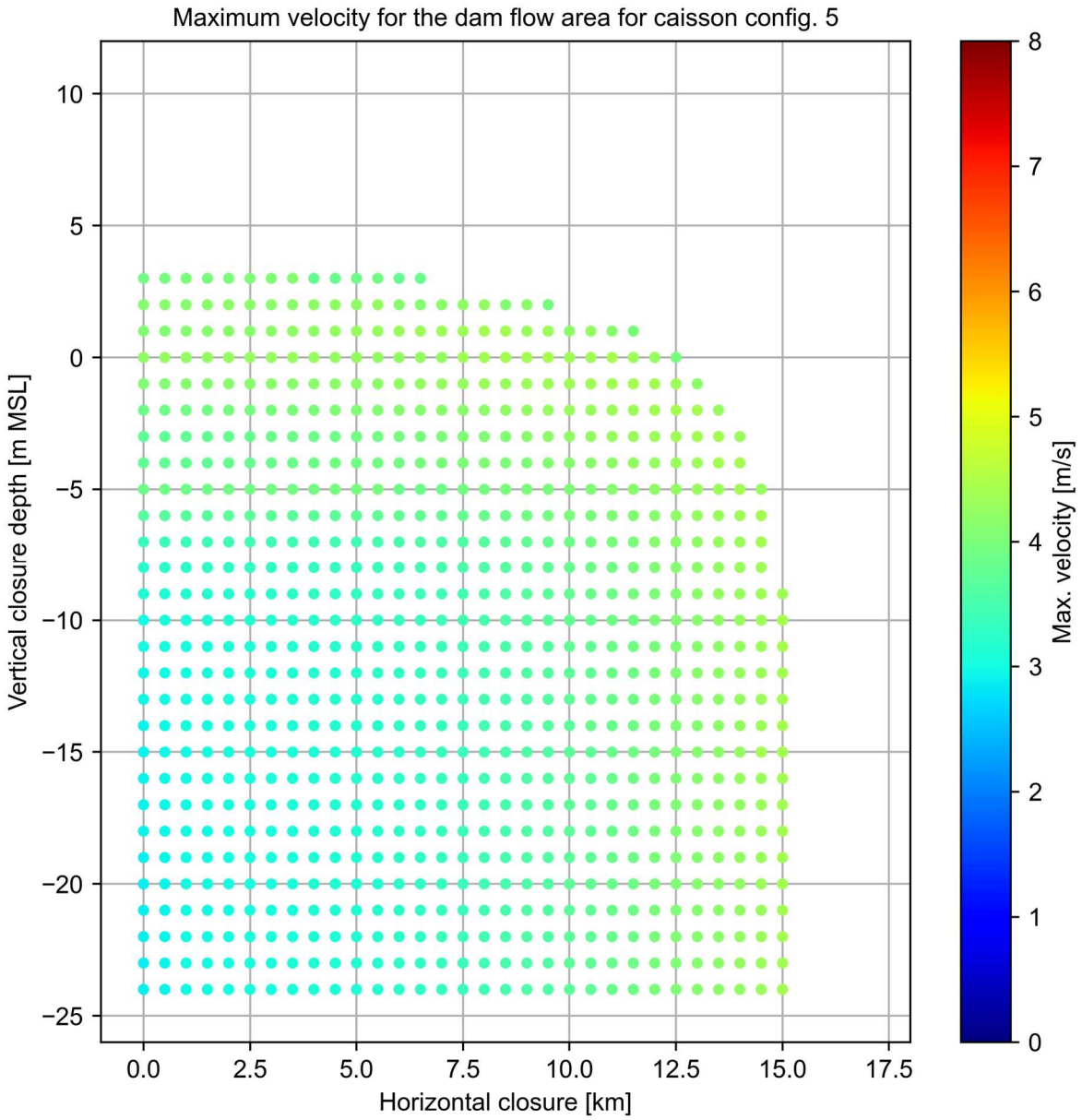


Figure 6-18: The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 5.

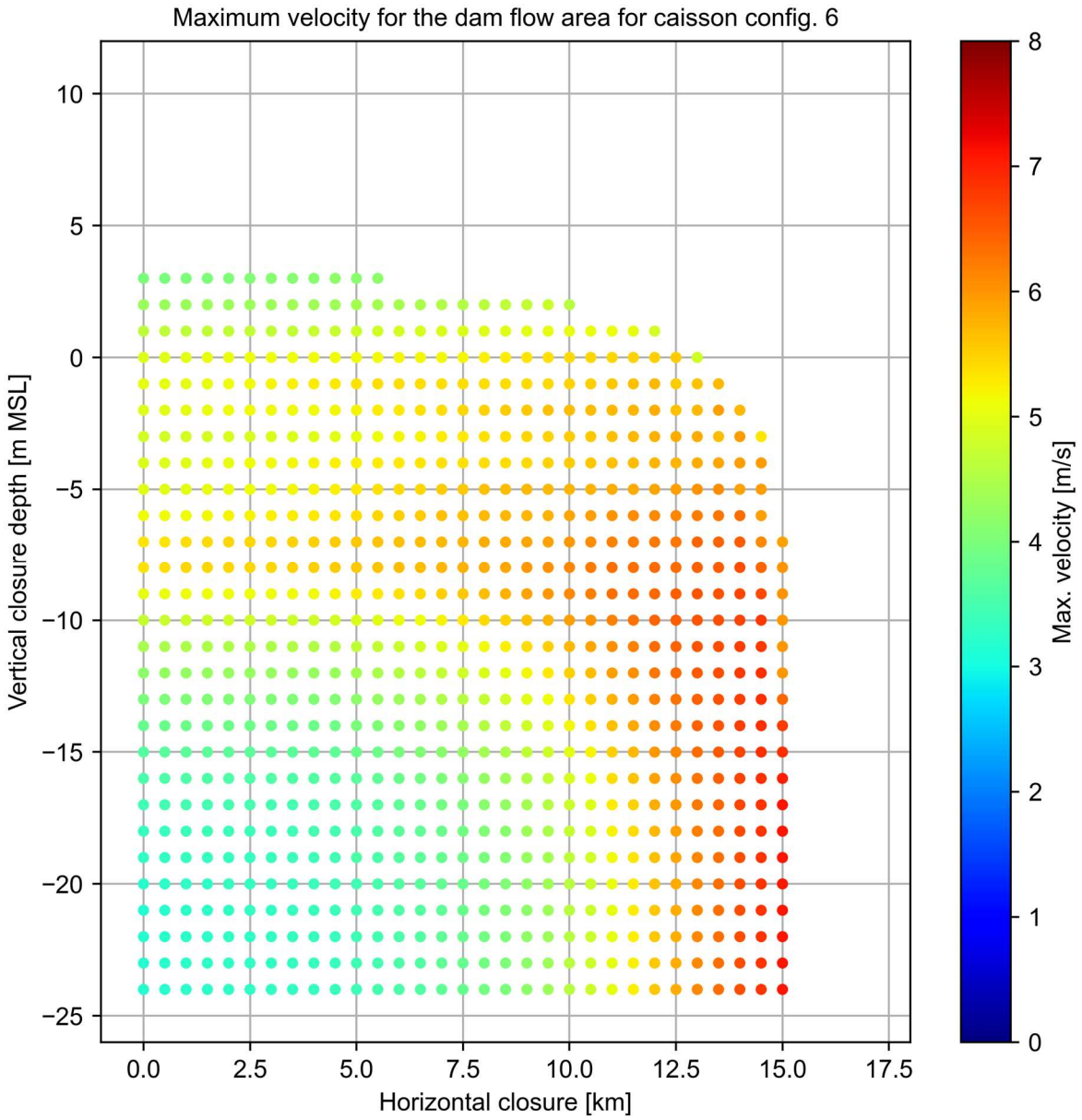


Figure 6-19: The maximum flow velocity for different horizontal and vertical closures for Concrete Structure 6.

To summarize the findings of Step 2:

- When the highest value of the maximum current velocity during the construction of the closure dam is not to be higher than 4 m/s, a total length of open structure of 15 km needs to be combined with 20 m high caissons. For that size of open concrete structure multiple horizontal and vertical closing strategies exist.
- When the highest value of the maximum current velocity is not to be higher than 6 m/s, a 10 km of open structure consisting of 20 m high caissons, or 15 km of open structure consisting of 15 m high caissons are required. For that size of concrete structure multiple horizontal and vertical closing strategies exist.
- The data generated in Step 2 is sufficient to determine the required stone sizes for different closure strategies.

6.3 Step 3: Optimization stage

The design process of the closure dam is a cyclic process rather than a linear process. It is therefore expected that some additional simulations are required for the design. Step 3 focusses on these additional simulations.

The placement of the caissons is a vital part of the closing strategy. The caissons are placed on a rock bund when the current velocity is sufficiently low for a sufficient amount of time. Such an event is referred to as a placing interval. The number of placing intervals strongly determine the construction time of the sudden closure structure.

To determine the number of placing intervals several simulations are performed for different horizontal and vertical closures. In Table 6-4 the different horizontal and vertical closure settings are summarized. For all scenarios are vertical closure up to – 12 m MSL is used. The level of -12 m MSL corresponds to a caisson height of 20 m. Based on the results of Step 1 and Step 2, the 20 m high caisson is expected to be most practical. The simulations are performed for the entire year of 2013.

Table 6-4: An overview of the simulations to determine the number of placing intervals.

Simulation number	Horizontal closure	Vertical closure	Figures
1	0 km	- 12 m MSL	Figure 6-20
2	5 km	- 12 m MSL	Figure 6-21
3	10 km	- 12 m MSL	Figure 6-22
4	15 km	- 12 m MSL	Figure 6-23
5	20 km	- 12 m MSL	Figure 6-24

The number of placing intervals is dependent on the minimum required duration and the maximum velocity. Because these exact boundaries are strongly dependent on the contractor multiple boundaries are investigated. In this specific analyse both the rising tides as the falling tides are investigate for the placing intervals. For safety reasons one might argue that only the rising tides should be considered. In that case the number of intervals can be divided by 2 because the tide is very symmetrical. The number of placing intervals is maximized as 2 intervals per tidal cycle i.e. one at rising tide and one at falling tide.

The results for the placing intervals are depicted in Figure 6-20 up-to Figure 6-24. As expected the number of intervals decrease when the required time increases. Also, the number of intervals increases when the maximum current velocity increases.

When the horizontal closure because larger the number of intervals decrease as well. This is because the current velocity increases when the horizontal closure becomes larger and resulting in less closing intervals.

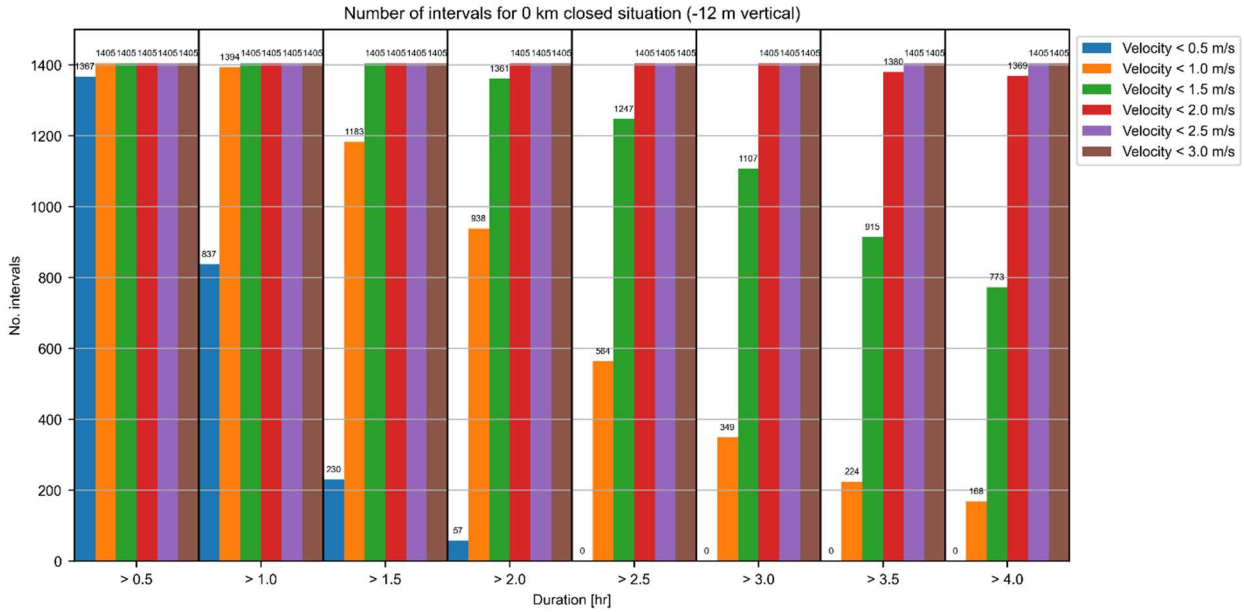


Figure 6-20: The number of closing intervals when there is no horizontal closure.

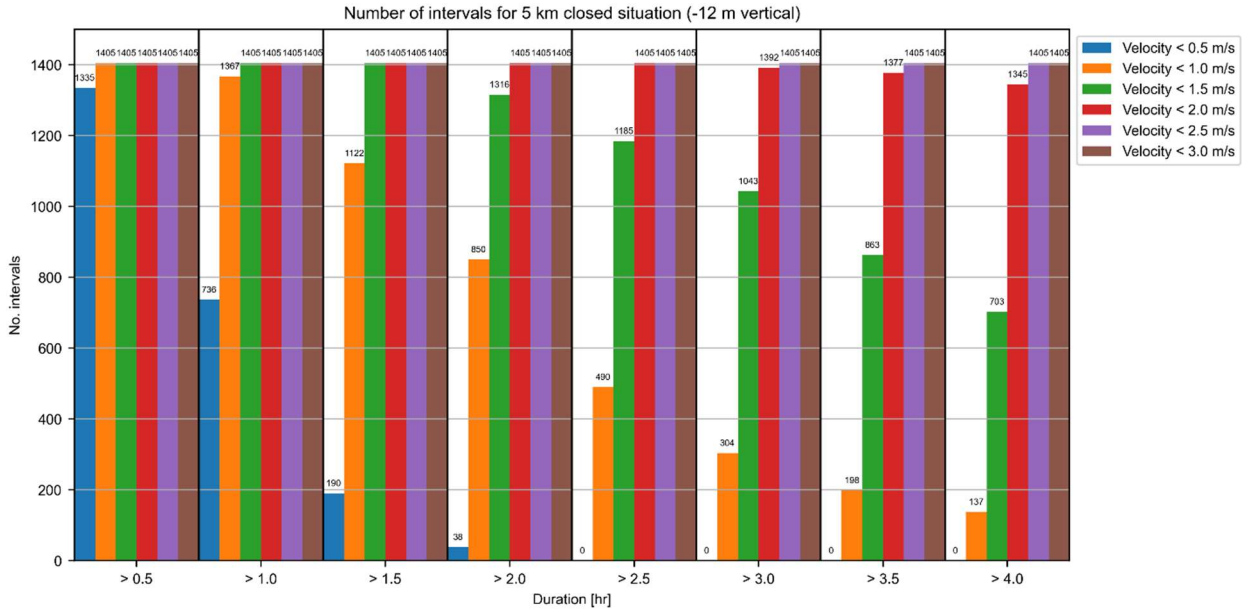


Figure 6-21: The number of closing intervals for a horizontal closure of 5 km.

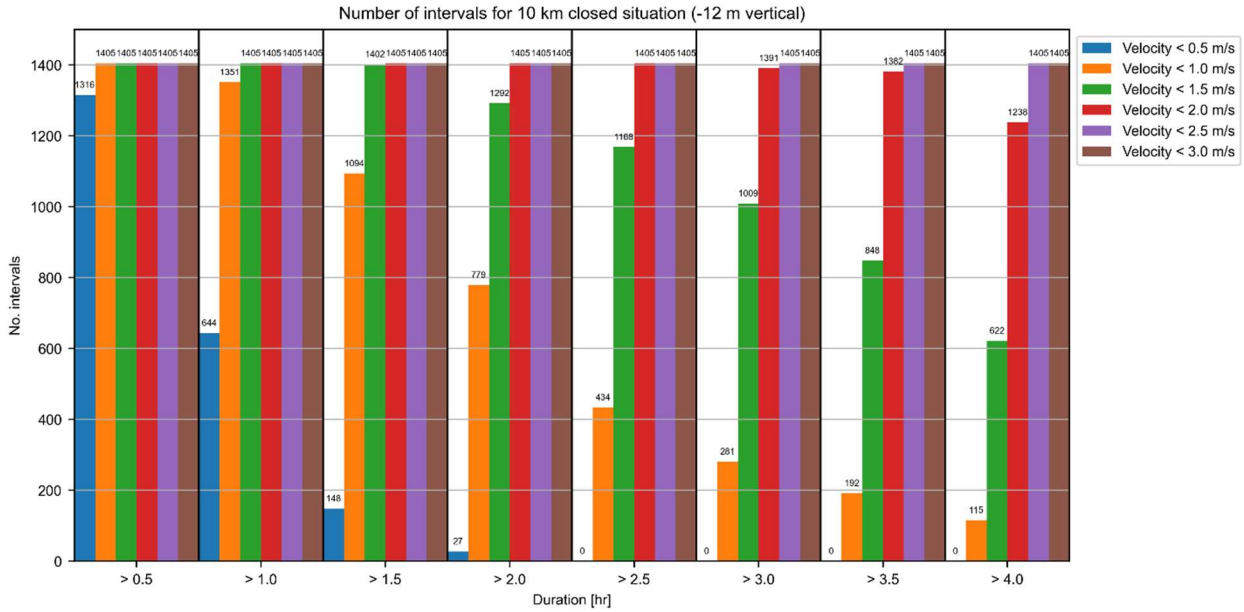


Figure 6-22: The number of closing intervals for a horizontal closure of 10 km.

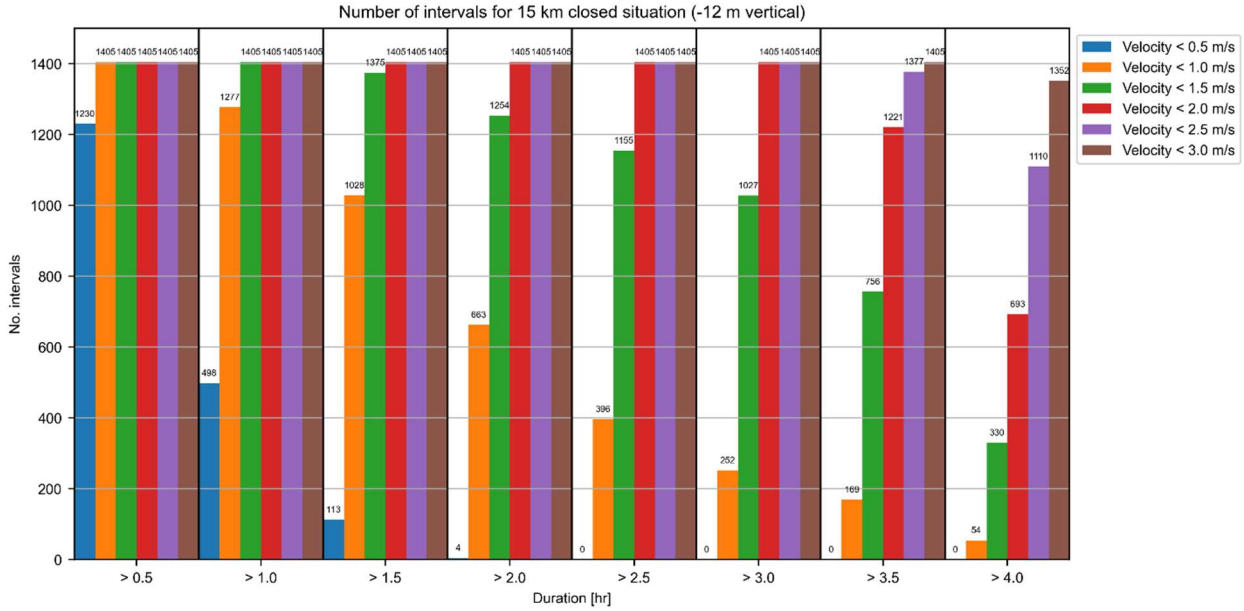


Figure 6-23: The number of closing intervals for a horizontal closure of 15 km.

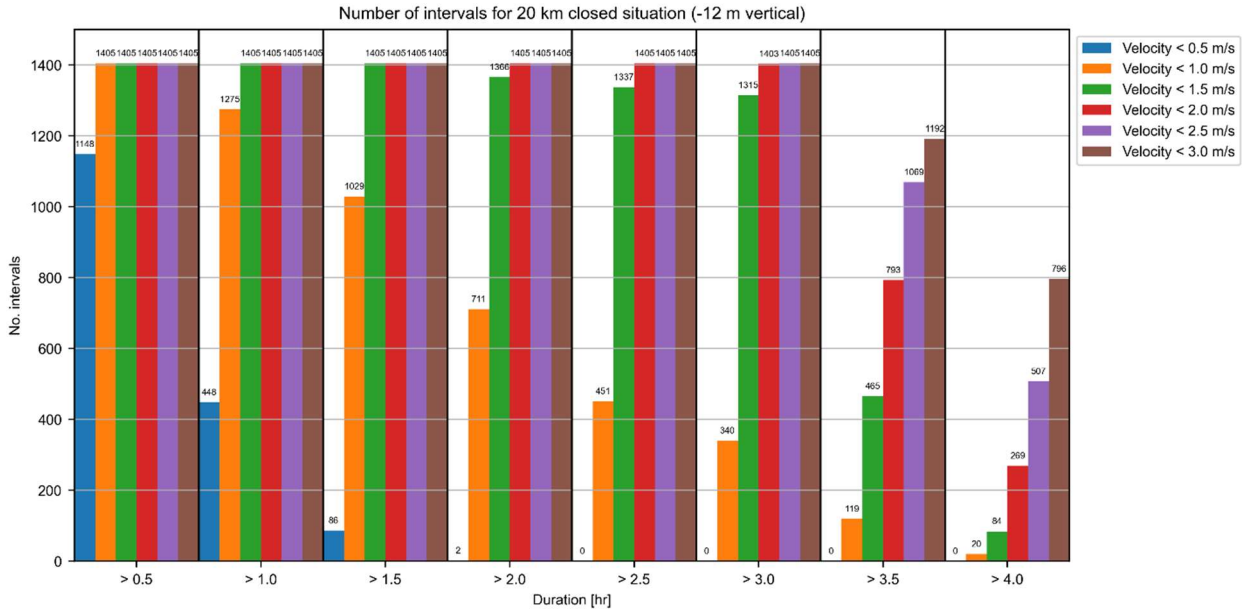


Figure 6-24: The number of closing intervals for a horizontal closure of 20 km.

7 Conclusions

The goal of the study at hand is to provide a sufficient amount of data to be able to determine the rock sizes for multiple closing strategies in Volume 2: Technical reports sudden closure. A total of six scatter plots and the related velocity matrices were created for an enormous number of closing strategies to be considered. The following main conclusions can be made using the results from this study:

1. A full horizontal closure (without any open structure) leads to a maximum flow velocity up to 10 m/s and is therefore not feasible. A full vertical closure (without any open structure) leads to a maximum flow velocity of 6 m/s and might be feasible.

2. Based on the results of Step 1 the following conclusions can be made:
   - The different tests with the model and the calibration show promising results and show that the modelled represent the physical reality accurately enough for this part of the design process.
   - The effect of the number of sections on the maximum flow velocity is minimum and can be neglected.
   - The maximum flow velocity is strong dependent on the length of the concrete structure. When the total length increases the maximum flow velocity reduces.
   - The maximum flow velocity is linear dependent on the length of the concrete structure.
   - The maximum flow velocity increases when the width of the gates is reduced. This influence is limited.
   - The maximum flow velocity decreases linearly with an increase of caisson height because a smaller area underneath needs to be filled up and the flow cross-section increases.
   - River discharge increases the flow velocity, but the effect is limited.

3. The following conclusions can be made based on the results in Step 2:

- When the highest value of the maximum current velocity is not to be higher than 4 m/s, 15 km of open structure consisting of 20 m high caissons are required. For that size of concrete structure multiple horizontal and vertical closing strategies exist.
- When the highest value of the maximum current velocity is not to be higher than 6 m/s, 10 km of open structure consisting of 20 m high caissons, or 15 km of open structure of 15 m high caissons are required. For that size of concrete structure multiple horizontal and vertical closing strategies exist.
- The exact closing strategy will depend on the constructability and the required stone sizes in each step. This this be the focus of Volume 2 report.

4. The following conclusions can be made based on the results in Step 3:
   - The number of placing intervals is strongly dependent on the maximum current velocity and the minimum required duration for a placing interval.

8 Recommendations and opportunities

The goal of this study was to provide sufficient data to determine a technical feasible design. In a more detailed stage of the design process a more detailed modelling study would be beneficial to reduce risks, reduce costs and increase the feasibility of the project. The following recommendations and opportunities are given:

- In this stage of the design process a 1D model was applied. The 2D current effects in proximity of the closure dam were therefore neglected. This current can be locally significantly stronger than the depth and width averaged velocities. It is therefore recommended to also model several closure strategies in a 2D model like Delft3D in a later design stage [5].
- The closure stages were limited by a 500 m horizontal or a 1 m vertical closure step. 2D closing strategies like closing a specific channel first or applying a vertical closure following the bed were not considered. The work of [4] has shown that this can reduce the required rock sizes. This could be an opportunity to reduce the costs of the closure dam.
- Both the accuracy of the numerical model and the closure strategy are heavily dependent on the correct representation of cross-section along the dam alignment. In this study the cross-section was used from the Delft3D model [5]. It is expected that the morphology along the dam is highly dynamic, and this creates large risks for the project. A better understanding of this risk can be reached by comparing multiple bed surveys. It is therefore recommended to perform a bathymetric survey every 6 months during a period of at least 2-3 years.
- The results of this study will be used to determine the scour during the construction of the closure dam and the bed protection using design guidelines. A detailed scour study could strongly reduce the required amount of bed protection and therefore reduce the costs of the project.
- The lifetime of a freshwater reservoir is strongly dependent on the amount of sedimentation accumulating in the reservoir which is likely to reduce its volume in time. This will strongly influence social and economic benefits of the project. It is thus generally recommended to perform a detailed sedimentation study to get a better understanding of the sediment accumulation in the reservoir and determine strategies to increase its lifetime and consequently the economic and social benefits. It is understood that an ongoing sedimentation study is being undertaken by L&T/NIOT for this purpose.
- The study at hand focusses solely on the flow modelling during the construction of the closure dam. The effects of other hydrodynamic conditions have not been considered. For example, the effect of the waves, wind and temperature. These conditions will influence the required rock size and the workability. It is therefore recommended to perform a detailed metocean and wave study. This study can be used to get more insight in the workability, and it might help to optimize the construction process reducing its duration.
- Towing of caissons to the dam location is subject to constraints of wave height and current velocity and water density. It is recommended to include assessment of workability conditions for towing of caissons in the above-mentioned metocean study.
- The spillway is not modelled explicitly in this model but is an integrated part of the concrete structure. For example, an open concrete structure length of 10 km might be 8 km of caissons and 2 km of spillway. By using the spillway during the sudden closure less open caissons are required which might lead to lower construction costs. In this example a 1 to 1 relation is assumed between the length of open caissons and the length of spillway. When the design is further progressed a more fundamental relation can be derived.

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Appendix A - Overview of the simulations of Step 1

Closure		Concrete structure		Caisson property		River discharge	Maximum velocity [m/s]
Vertical	Horizontal	Length [km]	Sections	Caisson height [m]	Gate width [m]		
-6, -8, -10	0, 5 km	2.5, 5, 7.5, 10, 12.5	1,2,3	15,20 m	10, 25, 50 m	none, medium, large	
-24 m MSL	0 km	0	0	none	none	none	2.33
-8 m MSL	0 km	10	3	20	25	none	3.46
-8 m MSL	0 km	12.5	3	20	25	none	3.27
-8 m MSL	0 km	7.5	3	20	25	none	3.67
-8 m MSL	0 km	10	2	20	25	none	3.46
-8 m MSL	0 km	5	2	20	25	none	3.96
-8 m MSL	0 km	2.5	1	20	25	none	4.28
-8 m MSL	0 km	10	3	20	50	none	3.42
-8 m MSL	0 km	12.5	3	20	50	none	3.24
-8 m MSL	0 km	7.5	3	20	50	none	3.64
-8 m MSL	0 km	5	2	20	50	none	3.93
-8 m MSL	0 km	2.5	1	20	50	none	4.27
-8 m MSL	0 km	10	3	20	10	none	3.57
-8 m MSL	0 km	12.5	3	20	10	none	3.40
-8 m MSL	0 km	7.5	3	20	10	none	3.78
-8 m MSL	0 km	5	2	20	10	none	4.04
-8 m MSL	0 km	2.5	1	20	10	none	4.33
-6 m MSL	0 km	12.5	3	20	25	none	3.78
-6 m MSL	0 km	10	2	20	25	none	4.21
-6 m MSL	0 km	5	2	20	25	none	4.99
-8 m MSL	5 km	10	2	20	25	none	3.73
-8 m MSL	0 km	10	2	20	25	medium	3.47
-8 m MSL	0 km	10	2	20	25	large 1	3.62
-8 m MSL	0 km	12.5	3	15	25	none	5.18
-8 m MSL	0 km	10	2	15	25	none	5.11
-8 m MSL	0 km	10	1	15	25	none	5.09
-8 m MSL	0 km	5	2	15	25	none	4.91
-10 m MSL	0 km	12.5	3	20	25	none	3.11
-10 m MSL	0 km	10	2	20	25	none	3.15
-10 m MSL	0 km	5	2	20	25	none	3.23
-8 m MSL	0 km	10	2	20	25	large 2	3.73

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REPORT

Volume 2: Technical Report

Kalpasar Closure Dam Feasibility Design

Client:	India Ministry of Earth Sciences, National Centre for Coastal Research (NCCR)
Reference:	BI5332-101-100-RHD-ZZ-XX-RP-Z-0003
Status:	Draft/01
Date:	3 February 2023



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Document title: Volume 2: Technical Report

Subtitle: Kalpasar Closure Dam Feasibility Design
Reference: BI5332-101-100-RHD-ZZ-XX-RP-Z-0003
Status: 01/Draft
Date: 3 February 2023
Project name: Kalpasar Dam
Project number: BI5332-101-100
Author(s): M.A. van Heereveld | G.J. Akkerman | S. Boersen | R. Jacobs

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Date / initials: 3 February 2023

Approved by: S. Kunta

Date / initials: 3 February 2023

Classification
Project related

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1 GENERAL

1.1 Introduction

Identified first in the 1970's for its tidal power resource and having been considered in different configurations over the years, at present the KALPASAR dam aims to provide a reliable source of fresh water for the drought-stricken state of Gujarat (Saurashtra in particular) by means of a 60 km dam between Bhavnagar and Aladar of which the centre 30 km concerns a closure dam in the Gulf of Khambhat (Figure 1.1). A reliable source of fresh water is established once the closure dam is constructed and the transition from sea water reservoir to freshwater reservoir has occurred. From earlier studies it was found that the construction of the closure dam and the saltwater transition will require 10 to 15 years before fresh water is available, pending further analysis beyond the scope of this assignment.

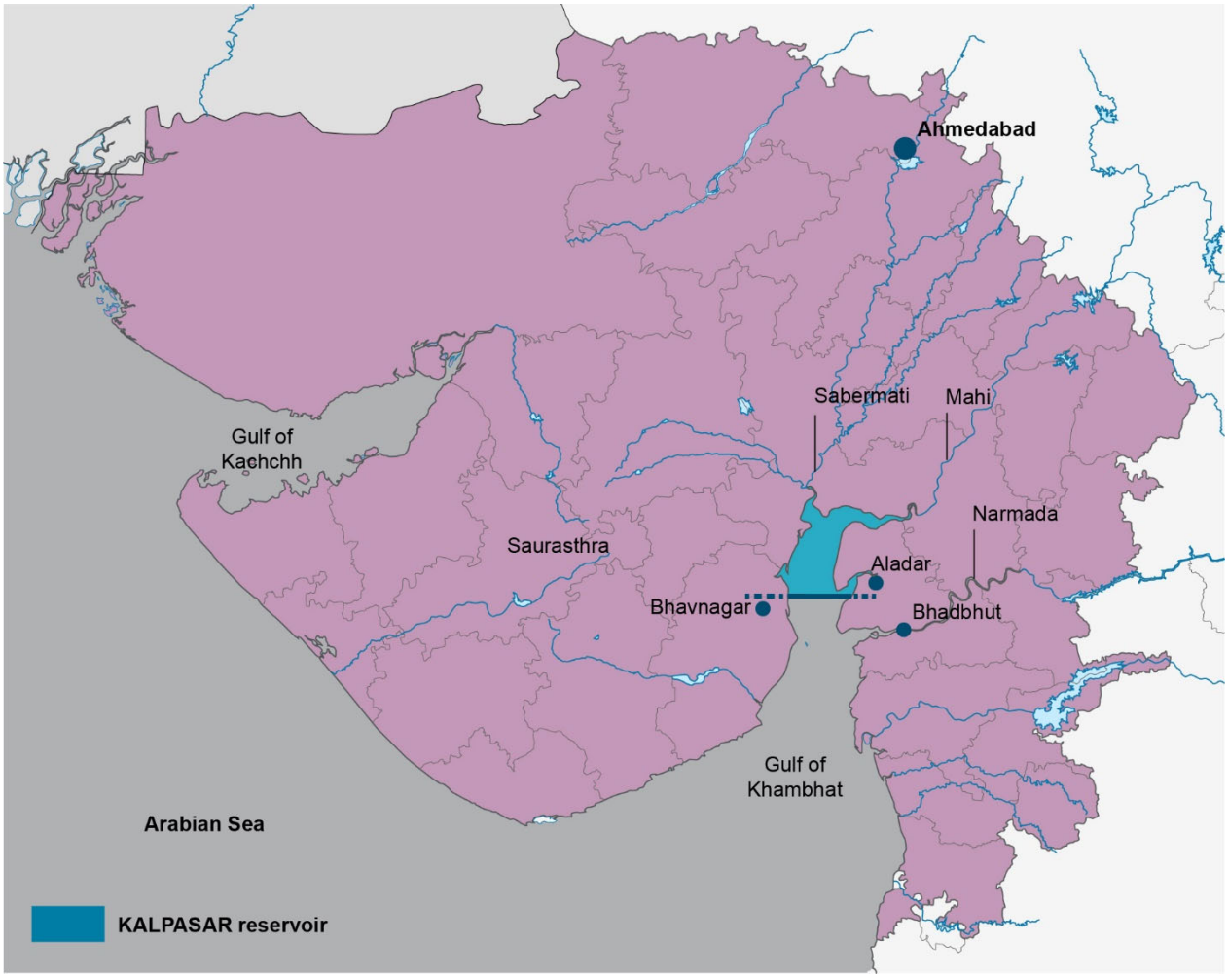


Figure 1.1 Selected alignment of the KALPASAR dam in the Gulf of Khambhat

In its present form, the KALPASAR project includes the 60 km dam connecting the Bhavnagar and Bharuch districts and a road/rail corridor, a freshwater reservoir of 7.800 million m^3 and a spillway to divert floods from the three major rivers flowing into the freshwater reservoir (Sabarmati, Mahi and Dhadhar). Once completed, a diversion from the Bhadbhut Barrage in the Narmada by means of a canal towards the freshwater reservoir is also considered by Narmada Water Resources, Water Supply and KALPASAR Department (NWRWS).

The KALPASAR dam is a huge undertaking considering the sheer volume of work and associated cost alone. Considering that during construction, severe site conditions must be accounted for such as, (i) 25 m water depth on average, (ii) a tidal range of 4 m to 10 m at Bhavnagar, (iii) poor soil conditions and (iv) monsoon seasons that take out 3 months of working time each year, makes the project even more challenging and costly. A delay in its completion – if construction risks are not appropriately mitigated – is likely to occur. Given the anticipated high cost of construction, the risk of delay before fresh water becomes finally available results in a viability gap that is difficult to mitigate, if at all. As such, KALPASAR dam is an unprecedented project.

The construction of the KALPASAR dam consists of two main stages: (i) the construction of the closure dam and (ii) the upgrading of the closure dam to the final (permanent) dam. The present feasibility study considers the feasibility of the closure dam only. This is the first and probably most important step in de-risking KALPASAR project. After all, once the closure dam is erected, the construction risk associated with upgrading it to its desired final form is dramatically lower, although still significant for the overall size of the project.

1.2 Main objectives and scope

The main objective and scope is to identify feasible closure strategies for the KALPASAR closure dam to allow for an overall feasibility assessment and/or design: an (optimised) closure strategy implies assuming construction methods, risks and mitigation, construction duration as well as costs, all of which are closely interlinked with the approach to KALPASAR dam itself and its feasibility. In recognition of this interdependency, rather than one optimised closure strategy, two distinct and feasible closure strategies are developed for consideration as part of the overall KALPASAR dam project. The distinct closure strategies may subsequently be used to facilitate an educated decision/evaluation between optimising for the shortest construction duration of the KALPASAR closure dam or the lowest cost and potentially a longer construction duration of KALPASAR closure dam within the context of the overall project: the optimum closure strategy can therefore be identified only as part of the overall KALPASAR dam project.

The two feasible closure strategies consist of gradual closure components and a sudden closure component, each of which have distinctly different risks associated with them, that ultimately determine the feasibility of a strategy. A key aspect of a feasible closure strategy is that the sudden closure of a substantial number of gates can be reliably closed at the ultimate moment of final closure: to this end, different types of gate and operating mechanisms will be considered in a quantitative risk analysis.

Further to the above main objective and scope, the permanent structures of KALPASAR dam and the design of the spillway gates is reviewed separately without considering them within the broader scope of feasibility of the KALPASAR closure dam.

1.3 Appreciation of the assignment

A closure strategy is the combined deployment of different closure components: the gradual components and the sudden component. Gradual closure components are the land-based / horizontal closure method or the seaborne / vertical closure method. Both methods use rock material dimensioned to remain in place despite (changing) flow conditions on site. The sudden component / method is a structural section in the closure dam which typically remains open until the very last moment to avoid further increase of flow velocities, before finally closing all structures in a relatively short but favourable window of site conditions. The sudden method is typically applied when rock material can no longer be (economically) deployed because flow velocities increase as construction progress is made and the flow cross-section narrows.

The KALPASAR closure dam is defined as the combined gradual and sudden closure components. Ground improvement works are not considered as part of the closure dam feasibility and are assumed to have been completed in line with requirements ahead of construction of the closure dam. Likewise, land reclamations for construction support and logistic purposes and/or (dry) docks for construction of structural closure components are considered to have been prepared appropriately so that only the actual closure works have to be erected within the target set for the duration of closure works.

The three closure components can be combined in many ways depending on any given set of requirements, leading to different closure strategies. As such there is not one single feasible closure strategy but rather a multitude of strategies that are technically feasible, though each will have a different (construction) risk profile. Whether risks can be (efficiently) controlled or not and whether residual risks are acceptable and affordable in broader terms determines the feasibility of a strategy. Risks are for instance technical risks, volume, price, construction risks or time-related risks.

The risk profiles of the main alternatives serve as ‘guide rails’ for the next stages of development. In the next stages it is anticipated that additional requirements are identified to ensure the KALPASAR project in its entirety is feasible and such requirements may potentially adversely affect the feasibility of the KALPASAR closure dam stage. Adhering to the ‘guide rails’ provided here helps ensure that where the closure strategy and components thereof need to be amended to make the overall project feasible, do not render the closure feasibility impossible.

The development of the KALPASAR closure dam is considered in a typical project life cycle. The project life cycle is divided into distinct stages separated by stage gate decisions. At a stage gate, conditions must be satisfied, and sufficient confidence must have been established to justify progressing on a next stage and secure / commit the associated funding (see Main Report).

The ultimate objective is that KALPASAR dam will deliver the expected (technical) performance, against minimised (institutional) risk and optimised (financial/economic) cost and revenue. This implies that at each stage gate in its project life cycle, risks of a technical, institutional, and financial / economic nature are addressed and considered adequately. However, the scope of the present assignment concerns the KALPASAR closure dam only and not the overall KALPASAR dam and appurtenant works. The scope is further limited to technical performance and associated risks and limited monetisation thereof only and institutional risks such as socio-environmental impact is – at present – not considered. Due to the nature of the KALPASAR project, it may be found that such other categories of risk render the project unfeasible, even if the closure dam component considered in the present assignment is deemed feasible.

1.4 (Historical) references

The 1998 pre-feasibility study on KALPASAR [1] provides a solid point of departure for the present assignment even though the southern alignment considered at the time is now abandoned for the present shorter and more northern alignment. The all-encompassing feasibility study, touching on a wide range of topics, provides context to the more limited closure feasibility presently considered. Since then, more studies have been performed that are valuable for the present project.

This pre-feasibility study builds on the technical study of 1996 but deals with a wide range of aspects of this development project: hydraulic and geotechnical conditions, dam design, tidal power, hydrology, land reclamation, irrigation, water supply, transportation, port development, fisheries, environmental impact and economical evaluation on a pre-feasibility level. Relevant for the present closure dam design are especially

1 Royal HaskoningDHV (1999), “Pre-feasibility study report on KALPASAR – Gulf of Khambhat Development Project”, D1568.21

the sections on hydraulic and geotechnical conditions and dam design. Notable conclusions at the time were:

- The economic feasibility of the proposed development seems viable, with the conclusion that combination of the creation of a freshwater reservoir and tidal power (5,000 MW) has the best development potential.
- The technical feasibility of the closure of the Gulf of Khambhat has been demonstrated on the basis of known and tried-out technologies.
- In spite of the above, the immense scale of the closure and conditions justifies the development of new and more economical techniques (which have not been considered in this study). Focus on more economic closure methods and materials (including caissons and auxiliary works).
- Maximum flow condition of 6 m/s is assumed and occurs at 10 km opening which thus should be closed by means of sudden closure.
- Bed protection will be required along the whole length of the dam.
- Sluice capacity (including Narmada inflow) occupies about 1.2 km of length at sill level MSL -11 m (65 sections of 17 m wide each) and includes 17 m high radial gates.
- Costs assessed at EUR 2 billion (price level 1998).
- Desalination time assessed at 4 years (in absence of tidal power).
- Seaside work harbours on both sides of the Gulf are required.
- Inclusion of tidal power is anticipated to be the best for the development objectives and its size can easily be optimised.
- Construction and logistical support hubs are required for construction with an approximate size of 500 x 700 m each.
- Present tidal currents reported in between 2 and 4 m/s.

Hydraulic and morphological impact of a closure dam in the Gulf of Khambhat [2] concerns a master's thesis at Delft University of Technology of R.T.C. Kersten, commissioned and supervised by Royal HaskoningDHV. As opposed to the pre-feasibility study, the thesis considers the present northern alignment of the KALPASAR dam, so excluding the Narmada River. The thesis deals with the hydro-morphological impact of the KALPASAR dam, with special focus on the seaward side. This is of importance for surrounding infrastructure, adjacent coastlines, natural environments and socio-economic activities. For this study, a morphodynamic model has been developed, based on a 2DH tidal propagation model from The Netherland's Deltares and the National Institute of Ocean Technology (NIOT) of India. In the calibration process, a new bathymetry has been introduced. By comparison of the situation with and without a closure dam, the morphodynamic effects could be assessed. It should be noted here that the morphological effects should be considered qualitative for the time being, as the predictive value of the model has not yet been ascertained. Most striking result is that the tidal range will be amplified strongly by the presence of the dam, e.g., the tidal range of 7.88 m will increase to 10.25 m (over 100 years even to 10.50 m). In the outward remaining estuary, the tidal dominance changes from ebb- tot flood-dominant. The changes will cause the estuary to start importing sediment and sedimentation near the dam will take place up to a distance of 40 km from the dam. As a consequence of the tidal amplification and the near field sedimentation, industrial maritime location may become prone to flooding, affecting e.g., the largest shipwrecking industry in the world at Alang, India's largest LNG terminal and the city of Surat. In addition, sedimentation in the approach channels to these locations may hinder their accessibility. Moreover, coastal erosion could take place over a considerable area.

Strictly speaking, the outcomes of the study are not relevant within the scope of the present study. However, the layout, functionality and operation of the dam may influence the phenomena as identified in the above

2 R.T.C. Kersten & Royal HaskoningDHV (2018), "Hydraulic and morphological impact of a closure dam in the Gulf of Khambhat", d.d. 02/07/2018.

study and hence mitigate the negative impacts. Mitigating measures focus on weakening of the reflection properties of the dam for the highest tides and may encompass amongst others: free or controlled inflow and outflow via open discharge sluices, provision for an overflow unit for highest tides over the dam body, the entering salt water to be disposed of via tidal power units and the discharge sluices during the subsequent ebb tidal cycle.

The thesis 'Optimising Closure Works, a case study on the Kalpasar Closure Dam' [3], has been set-up against the background of optimising the closure sequence of closure works. Such optimisation usually is extremely time-consuming and, as a consequence, possible optimum scenarios are missed. To this end, a parametric optimisation tool has been developed in order to be able to perform a first-order evaluation of possible closure strategies, efforts and costs. For the case study this tool has been applied for the Kalpasar Closure Dam. More specifically, the optimisation tool includes the required dam materials, bed protection lay-out and material and considers required equipment for six pre-programmed strategies. The hydraulic boundary conditions are derived from a multi-sectional storage model with separate modelling of tidal flats. 'strategic' input parameters are the closure method (horizontal, vertical or sudden), execution phase and construction capabilities to be defined for all sections. The tool calculates the closure sequence in time and the costs incurred in each strategic scenario. Optimisation is achieved when minimum costs are attained.

For KALPASAR closure dam, the optimisation tool (involving the storage model approach) has been calibrated with previous research with the 2DH-model from which an 'artificial contraction factor' has been determined. Relevant outcomes are:

- The complex bathymetry cannot properly be modelled with single cross-sectional models.
- Indications on optimised closure: first closing the tidal flats horizontally by forward dumping of rock, while closing the channels up to some 40% of their depth by dumping ships. Next the dam can further be closed vertically (or caissons can be placed) by a cableway or bridge system.
- This tool is highly useful for further usage in our project (after some development). The tool and background knowledge has been included in this study (Mr H. de Jong, at the time graduate and developer, is working for Royal HaskoningDHV at present and participating in the team for the present assignment).

1.5 Present context

Regional (infrastructure) developments related to KALPASAR project are the construction of the barrage at Bhadbhut and the Narmada Canal. Originally inside the KALPASAR reservoir, with the selection of a more northern alignment, the construction of the Bhadbhut Barrage allows to divert water by means of a canal from the Narmada towards the KALPASAR reservoir. Tidal power is no longer part of the scope of the KALPASAR dam and therefore not further considered in the present feasibility study for the KALPASAR closure dam.

Arup report Dyke Closure Design Section 24.0 [4]: A major change for the KALPASAR dam occurred in 2009 when it was decided to change the alignment towards a more northern position when compared to the alignment taken in the Haskoning feasibility study in 1998. The northern alignment implies that outflow of the Narmada River at the seaside of the KALPASAR dam. Furthermore, the channel depths for closure are smaller in the northern alignment and the length of the closure dam is less. In all, the feasibility of the closure may be expected to be favoured for the new alignment. An early update study for the new alignment indicated a required sudden closure length of 7.4 km. Moreover, a geotextile block mattress bed protection was anticipated to be placed along the dike alignment after 2 m of dredging. The mattress to be protected

3 De Jong & Royal HaskoningDHV (2018), "Optimising Closure Works, a case study on the Kalpasar Closure Dam".
4 ARUP (2022)

by 0.5 m rock dumping in the shallow parts and 1.0 m rock dumping in the deeper parts. The ARUP study of 2022 concerns the design of the final KALPASAR dam, however ignoring altogether the construction process and closure stage in particular.

NIOT studies: Various subsequent studies were carried out next, supported by further detailed numerical hydraulic studies carried out by NIOT in order to refine the construction methodology and its impacts. The modelling considered 30 km of closure dike length and 14 different closure scenarios. A distinction was made in Phase 1: construction of 3 working islands and Phase 2, closure of the 4 remaining channels. Outcomes of maximum flow velocities were presented in tables for the different scenarios.

Client Powerpoint presentation: Construction of KALPASAR dam: The Powerpoint presentation provided by the Client, indicated the anticipated final dam sections (caisson location and outside of caisson location), the design parameters for closure and bed protection design, and alternative sequences for closure of the 4 channels, showing the maximum flow velocity in each closure gap still open as the major design parameter. Based on the analysis, final closure of Gap C (Gaps A, B, C and D representing the north to south channels) was considered most favourable. As the maximum flow velocity, without any further constriction of Gap C already exceeds the value of 6 m/s, caisson closure has further been taken for Gap C. Subsequently, different closure methods are being presented in terms of equipment and handling. It should be noted here, that the choice for complete closure of gaps A, B and D, will lead to difficult conditions for closure of gap C. This is reflected in the minimum stone size of 1 to 2 m for the construction of the sill, for the caissons from the bed level up. Consequently, the stone size on the bed protection will also be very large (requiring individual placement of the stones which be very costly or even unfeasible). An important finding is that the construction of the working islands did not significantly increase the flow velocities. Relevant outcomes are:

- The outcomes are highly relevant as the studies include the new alignment.
- In spite of the more favourable alignment, the outcomes show that the closure is still unprecedented in terms of difficulty of closure. Therefore, the usage of caissons for sudden closure, as emphasized in the Haskoning feasibility study of 1998, also applies here for the final closure gap.
- Full closure of the closure gaps, prior to leaving gap 3 fully open, as investigated does not seem optimal, as regards of current velocities already exceeding 6 m/s prior to any closure of gap 3 (Note: this is our interpretation; this interpretation has not been mentioned as such in the above studies).

1.6 Overview of scope of assignment

The scope of the present assignment consists of two aspects, namely (i) the Design of Closure Methodology for KALPASAR closure dam, consisting of review of existing closure methodology reports and study various options for closure of the Gulf of Khambhat and an optimised closure methodology and design the same and (ii) the design of gate and hydraulic mechanical system. This scope has been executed and documented as described in the following sub-chapters.

1.7 Guidance for the reader

The execution of the study is documented in a Detailed Project Report with four supplementary Technical Volumes as annexures, structured as follows:

Detailed Project Report (DPR)
The Detailed Project Report (DPR) or Main Report takes the results from the technical report on the KALPASAR closure dam feasibility (VOLUME 2) to assess and evaluate risks to account for when considering the alternatives identified. Risks addressed are amongst others volume risks, price risks, time related risks and the failure mode analysis as (key) technical risks and mitigated in their respective designs.

The DPR includes an executive summary, key issues and risks to consider and recommendations on the way forward.

VOLUME 1: Technical Report on Hydrodynamic Modelling of KALPASAR Dam Closure
Volume 1 introduces the closure methods and strategies, and describes the hydraulic analysis based on thousands of simulations with the one-dimensional model hydraulic model, conducted to determine current velocities occurring during the closure for different closure options.

VOLUME 2: Technical Report on Closure Dam Feasibility (THIS REPORT)
From a thorough analysis of site conditions, a design basis for KALPASAR closure dam is determined to provide an unequivocal and solid basis for (i) development of distinct and feasible alternatives and (ii) risks to account for in the feasibility analysis presented in the Main Report. Gradual and sudden closure components are designed according to the design basis and used as building blocks allowing for evaluation of different closure scenarios in a construction model that provides insight into cost and duration of construction. From the evaluation of scenarios, the distinct closure strategies are derived and evaluated for input the feasibility analysis. The construction model used results in a theoretical design that is subsequently translated into a practical design for (i) illustrative purposes and (ii) to serve as a basis for the Bill of Quantities and Cost Estimates in Volume 4.

The approach and methodology for feasibility of the closure dam is presented in Chapter 2. In Chapter 3 site conditions are presented and evaluated. A design basis for KALPASAR closure dam is subsequently identified and presented in Chapter 4. In Chapter 5, the gradual closure components are presented, whereas the closure element is presented in Chapter 6. Different closure scenarios are subsequently evaluated and presented in Chapter 7, allowing for determining of the two alternatives optimised for either cost or duration of construction in Chapter 8, where the reliability of the closure element is also evaluated and optimised. This is then the basis for the feasibility analysis in the Main Report.

VOLUME 3: Technical Report on Design Review of Permanent Structures KALPASAR Dam and Design of Spillway Gates
Volume 3 contains the identification and assessment of options for the spillway gates, concluding in a conceptual design of the preferred option, a cost estimate and an operations and maintenance plan. In addition, a review of the KALPASAR dam structures in their permanent form, including the spillway, is provided as additional feedback on the designs by other consultants provided by the client.

VOLUME 4: Technical Report on Cost Estimate for KALPASAR Closure Dam
Volume 4 contains specifications, a Bill of Quantities (BOQ) and an associated cost estimate for the KALPASAR closure dam alternatives.

2 WORKFLOW

Our approach and methodology is presented in Figure 2.1. In order to arrive at a conclusive judgement on feasibility of the KALPASAR closure dam construction, steps A through H are followed, starting with an assessment of site conditions (step A) to allow for setting up a hydraulic model to simulate the closure alternatives (step B). The hydraulic model is then used to determine the hydraulic boundary conditions that will determine the geometry of the closure components (gradual horizontal, gradual vertical and sudden) for different closure scenarios in step C. Once the main geometry is known, a target construction time for the closure dam is set, considering overall construction time estimates from previous studies for the overall KALPASAR dam (step D). Combined with the site conditions from step A (annual working and production windows mainly), practical construction considerations are determined in step E to determine a required production capacities and logistic requirements for each alternative. This allows to prepare outline designs for each alternative in step F which are subsequently assessed in terms of their feasibility in step G. The feasibility of the closure dam is then considered in the broader context of the feasibility of the KALPASAR dam in its entirety to bring focus and allow prioritisation of project risks in the next steps, which is needed to efficiently and effectively de-risk the project.

The main (theoretical) geometry of the closure components is determined in step C, e.g., the length of horizontal closure, height of vertical closure and remaining gap for sudden closure. In alternatives I and II, the gradual closure components are emphasised by means of applying a maximum velocity to identify their (largest possible) main geometries. The geometry of the remaining opening must then be closed by means of sudden closure as flow velocities would otherwise require rock sizes that are no longer practical.

Taking into account overall project considerations (step D), a target construction time for the closure dam can be identified. This is necessary to have the feasibility of the closure dam not adversely affecting the overall KALPASAR dam project's feasibility: after all, if the closure dam would require 10 to 15 years to construct in an extreme case, the overall project might take even longer still which potentially hampers overall (economic) feasibility of the KALPASAR project, so a limitation must be applied.

After the target construction time is set it is possible to set a target production capacity per closure component and identify associated logistical requirements, material availability and sources etc (step E). The main geometry determined from step C is also divided into sub-stages, taking into account annual working / production windows, so allowing to determine the number of intermediary construction fronts for each component throughout the construction of the closure dam and optimisation of production of sudden closure elements.

Outline designs for each alternative are prepared in step F, including a limited sensitivity analysis to identify the impact of present design uncertainties on the closure dam (practical) geometry itself (i.e., the closure dam cross-section and not the (theoretical) main geometry in front view (step C). These designs are then evaluated in terms of feasibility in step G and provide the basis for the Bill of Quantities and cost-estimates (Volume 4).

The results of the feasibility assessment for the closure dam are then considered in the broader context of the KALPASAR project in step H, thereby noting that this is limited to (i) interfaces with the feasibility of the closure dam and (ii) other key risks known to us from previous involvement with the project and experience since elsewhere. This includes recommendations for effective de-risking of the project and efficient progress through the next stage gate by means of a way forward for consideration of the client as previously stated.

For the feasibility assessment, two main alternatives are presented: the alternative with the shortest construction time and the alternative with the lowest cost. Both alternatives use the optimised gate type and operating mechanism for the most reliable sudden closure performance. The two alternatives which are both technically feasible are evaluated in the Main Report. They will specifically allow to highlight key aspects and risks to consider when pursuing either alternative in the next stage of development of the KALPASAR dam project as a whole.

Quantitative and qualitative risk analysis are distinguished. The quantitative risk analysis is applied for the gradual and closure components and a qualitative risk analysis is applied for the sudden closure component only. The main difference between the two is that the former is quantified in terms of impact and to some extent (quantified) mitigative interventions, but not in terms of failure probability. Whereas the latter is not quantified in terms of impact but in terms of failure probability only (e.g., whether it is feasible to reliably perform a successful sudden closure). This is deemed fit for the present purpose and this will be provided in the Main Report.

The qualitative risk assessment for gradual and sudden closure components consists of identification of a TOP 5 technical risks, volume risks, price risks and time related risks in the Main Report. In so far as possible, risks are also monetised there or expressed in potential delay of construction. Thereby allowing the client to develop an educated preference for either alternative to consider when progressing onto the next stage.

The quantitative risk assessment for the sudden closure concerns a failure probability analysis for different types of gate in the sudden closure elements. For each type of sudden closure element, the individual failure risk is determined first. The failure risk is then extrapolated towards a combined failure risk (parallel closure) for the number of sudden closure elements needed to close the remaining gap determined in each of the three alternatives.

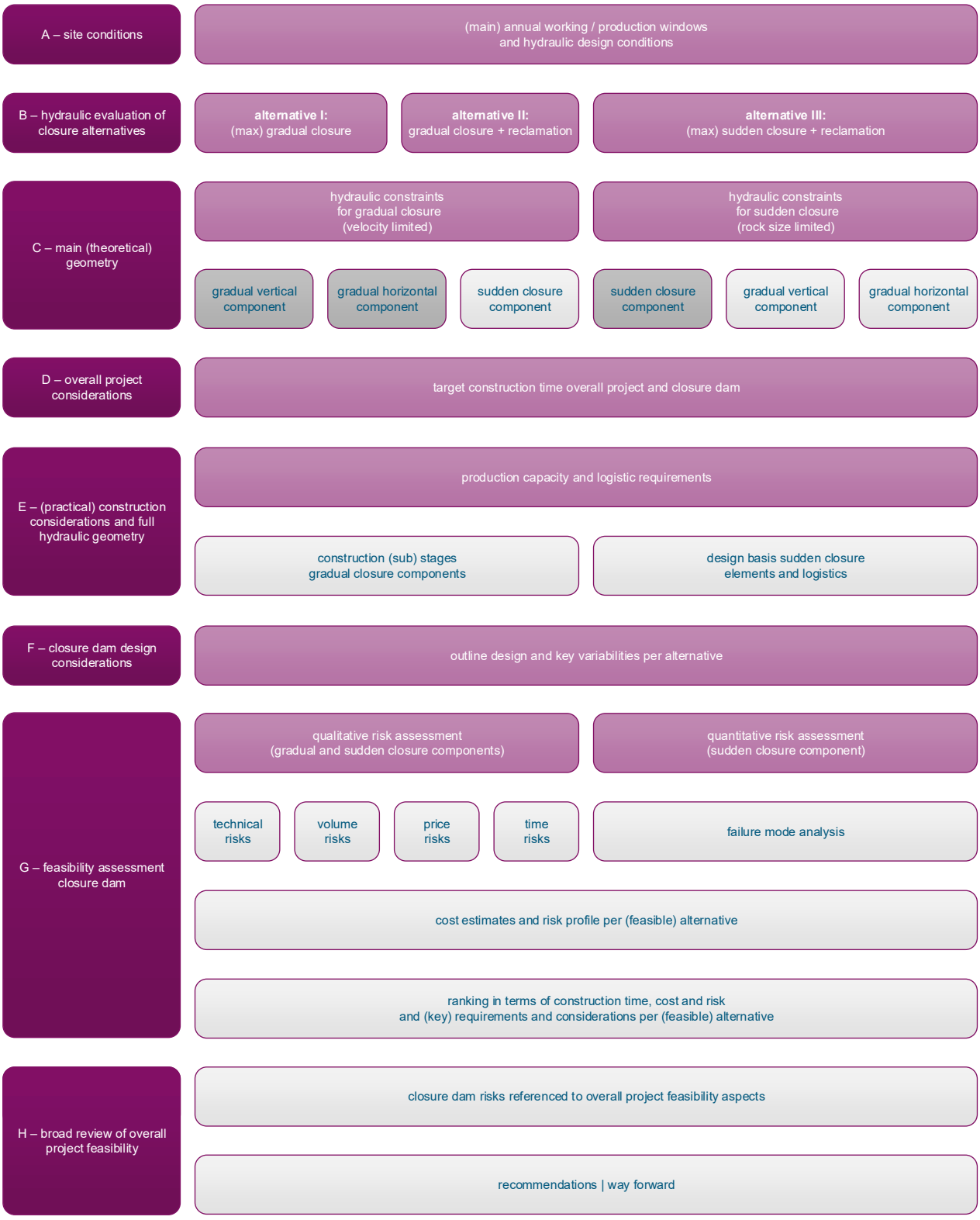


Figure 2.1 Workflow for the identification of alternatives and feasibility study

3 SITE CONDITIONS

In this Chapter, relevant (hydraulic) design conditions are presented. It should be noted that the site conditions presented here are taken from existing (recent) studies. Conditions presented are deemed representative for evaluating the feasibility of the KALPASAR closure dam and to identify key risks and considerations to account for in the next stage of development. Purpose-specific surveys and (probabilistic) analysis will be necessary in the next stage of development, aiming to improve relevance and accuracy for conditions on the actual site of the closure dam (during construction). Purpose-specific surveys and studies are at least a two-dimensional hydraulic modelling of construction progress and wave studies. The construction model and construction duration / progress found can be used as a basis for hydraulic modelling in the next stage.

3.1 Tidal characteristics and conditions

Tidal characteristics have been determined from a dataset from Hazira during 2013. The complete dataset is shown in Figure 3.1. It concerns a semi-diurnal tide with the tidal range varying from 8.28 m during spring tides and 1.53 m during neap (Figure 3.2). The average tide is 4.78 m. Further tidal characteristics are presented in Table 3.1.

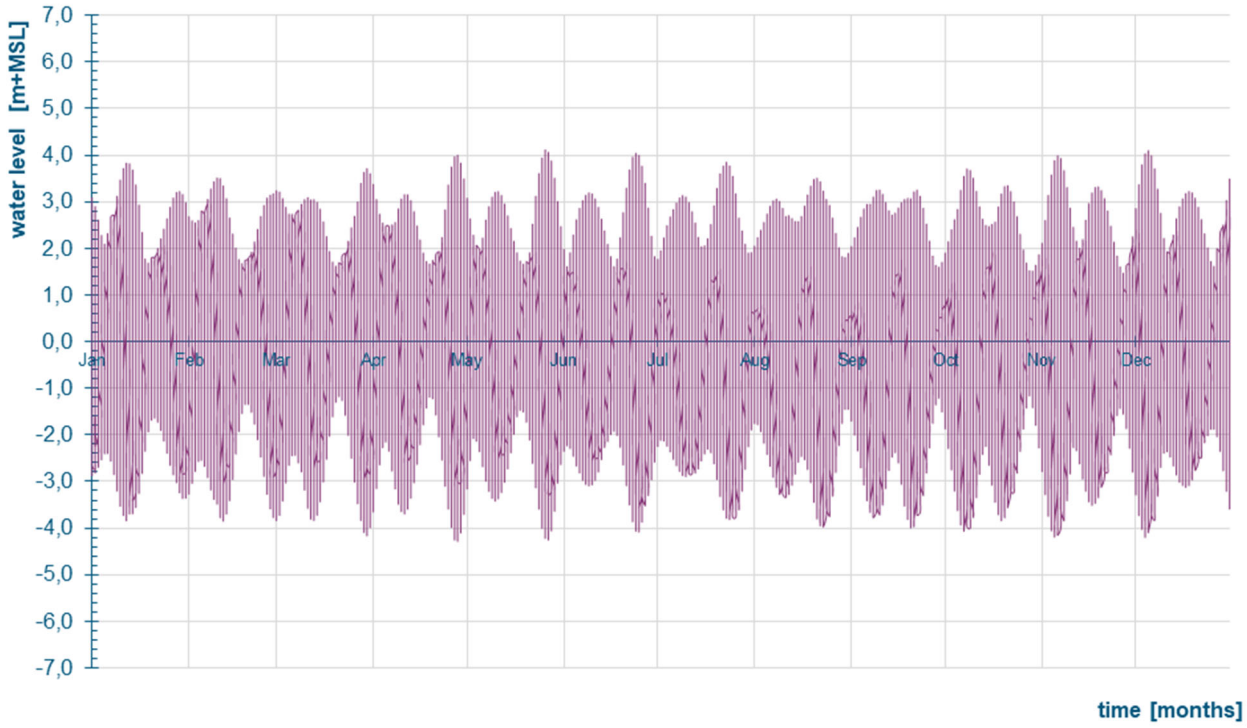


Figure 3.1 Annual tidal variation at Hazira (2013 dataset)

Using the one-dimensional hydraulic model, the tidal characteristics of at the location of KALPASAR closure dam have been determined. This is shown in Figure 3.3. At the location of KALPASAR closure dam, the tidal range is amplified: the tidal range in case of a spring tide is 10.14 m, versus 8.28 m at Hazira. The tidal range during a neap tide is approximately 3.06 m and an average tidal range is 6.37 m at the location of KALPASAR dam (1.53 m and 4.78 m respectively near Hazira). For further details regarding the hydraulic model, reference is made to Volume 1.

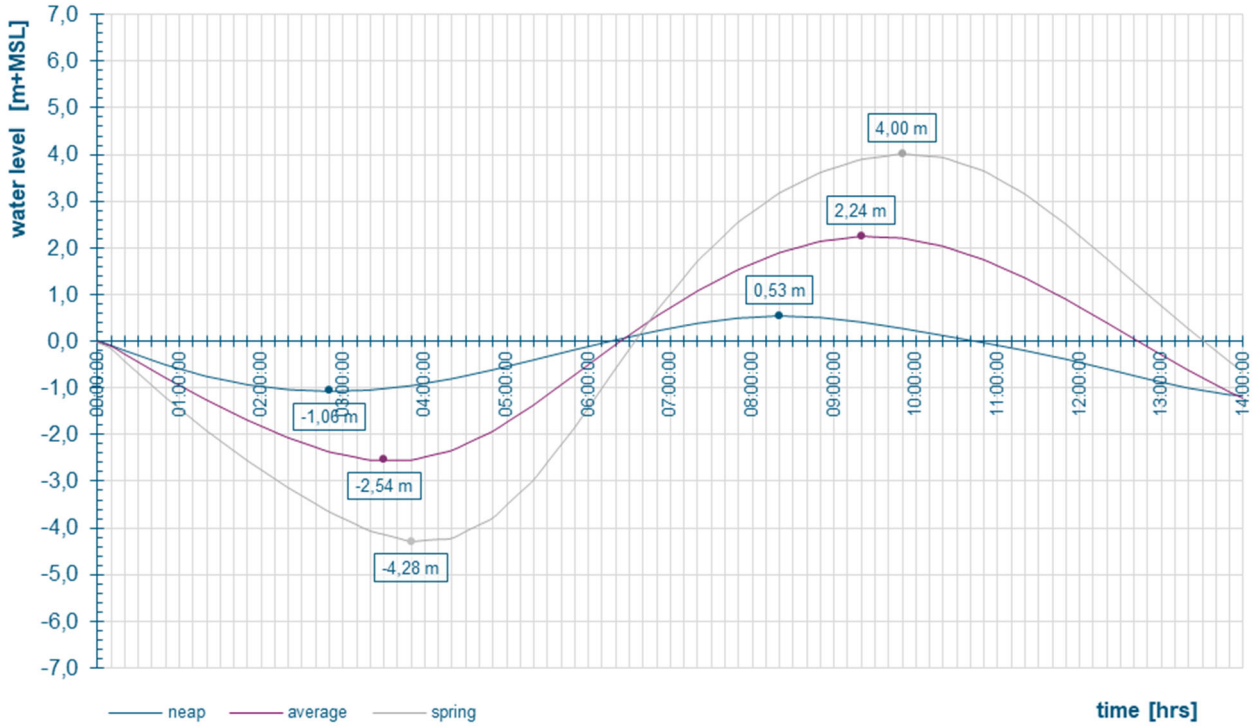


Figure 3.2 Neap, average and spring tide as observed at Hazira [5]

Table 3.1 Overview of tidal characteristics

		tidal datums relative to CD [m]
HAT	Highest Astronomical Tide	+10.80
HHWS	Highest High Water Spring	+10.20
MHWS	Mean High Water Spring	+8.80
MHWN	Mean High Water Neap	+7.10
MSL	Mean Sea Level	+5.10
MLWN	Mean Low Water Neap	+1.80
MLWS	Mean Low Water Spring	+0.90
LLWS	Lowest Low Water Spring	+0.70
LAT	Lowest Astronomical Tide	-1.00

The hydraulic model provides insight into the flow velocities in the present situation without the KALPASAR closure dam: profile average flow velocities typically vary between 1.9 m/s and 2.3 m/s. Flow velocities will increase as (gradual closure) construction progresses (see Paragraph 4.3).

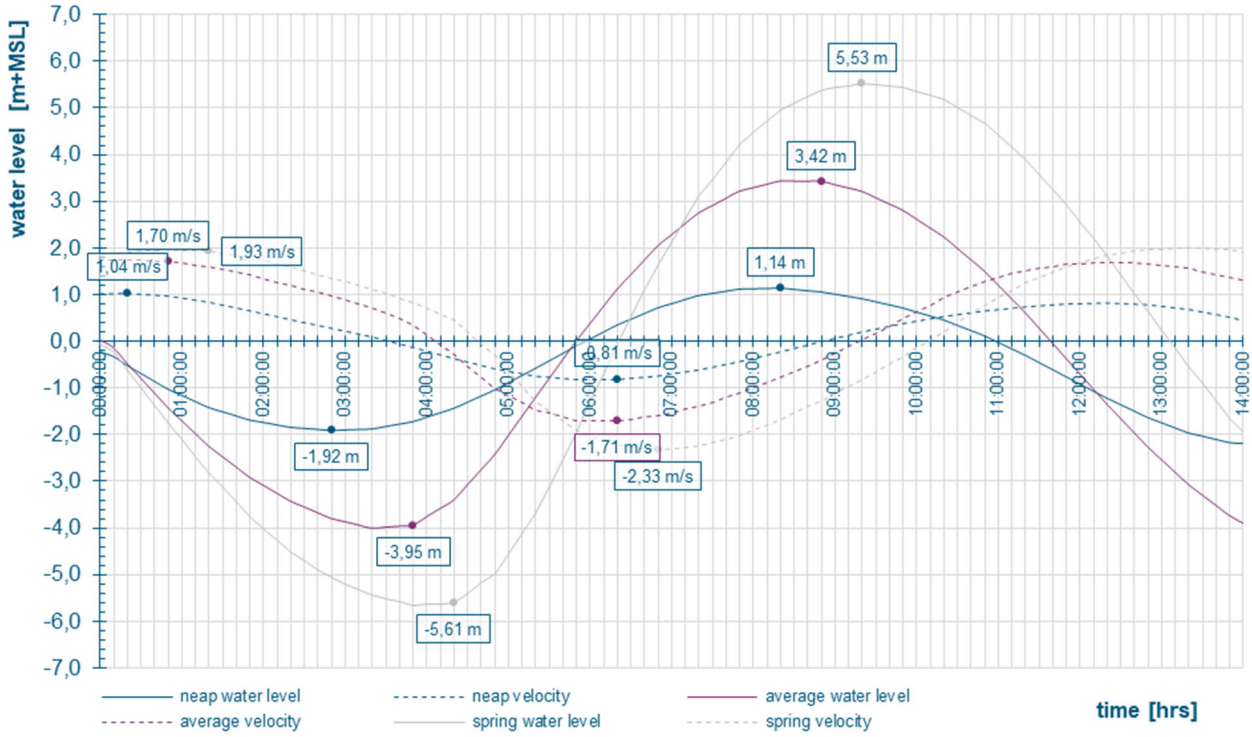


Figure 3.3 Computed neap, average and spring tide and associated profile average velocity near dam location (pre-closure)
Results obtained from indicative one-dimensional simulations

3.2 Wind conditions

To obtain insight into wind conditions at the site of construction of KALPASAR closure dam and how these conditions may affect construction, use is made of [5]. This study was prepared for a nearby offshore wind farm, some 40 km away from the construction site, south of Mahuva on the Saurasthra peninsula. Wind conditions are taken to be representative and fit-for-purpose of determining feasibility of the KALPASAR closure dam, pending further investigation in the next stage of development.

Table 3.2 shows the average and maximum wind speed U_{10} variation per month and the respective standard deviation. From the table follows a slight elevation of wind speeds only during monsoon season, from June to September with wind speeds elevated already during pre-monsoon in May. Post-monsoon wind speeds are typically lowest, notwithstanding cyclones which may not have been included in the analysis used.

Table 3.2 Wind speeds (U_{10} [m/s]) per month and yearly [5]

	jan	feb	mar	apr	may	jun	jul	aug	sep	oct	nov	dec	year
max	10.14	9.59	9.28	10.24	12.40	12.57	13.08	13.73	13.44	8.40	8.93	10.33	13.73
mean	5.55	4.84	4.40	4.61	6.07	7.24	7.69	6.67	5.22	3.43	4.45	5.24	5.46
stdev	1.72	1.65	1.36	1.39	1.58	2.14	1.89	2.26	2.29	1.55	1.55	1.59	2.15

Table 3.3 Directional statistical data of wind speeds (U_{10} [m/s]) [5]

	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°	year
max	10.33	10.24	9.56	6.98	6.78	8.72	11.56	12.19	13.73	11.65	9.63	9.12	13.73

5 COWI (2017), “First Offshore Wind Project of India (FOWPI) – Metocean Study”, A073635-014-001, d.d. 07/09/2017.

	0°	30°	60°	90°	120°	150°	180°	210°	240°	270°	300°	330°	year
mean	5.05	5.44	4.48	3.22	2.76	3.23	5.00	6.30	6.93	5.07	3.81	4.07	5.46
stdev	1.66	1.64	1.55	1.27	1.61	2.19	2.70	2.21	2.19	1.86	1.28	1.39	2.15

Table 3.4 Extreme value analysis of wind speeds 2010 – 2014 (U_{10} [m/s]) [5]

	return period [yrs]		
U_{10} [m/s]	1	5	10
central estimate	12.6	13.9	14.4
stdev	0.4	0.5	0.6

3.3 Wave conditions

A wave study is not part of the present scope. In absence of a wave study to determine hydraulic design conditions throughout the construction of the KALPASAR closure dam, NOAA offshore wave data is used. This is a conservative approach as offshore wave heights are expected to be higher than the (nearshore) wave heights at the dam site. Of the three nearest NOAA data points, point C is used as point A is nearest the NOAA model boundary where results are typically less reliable. Based on expert judgement and available references, the wave heights and return periods can be correlated to expected wave heights at the KALPASAR closure dam site during the various key stages of construction (Chapter 4). This approach is considered fit for the present purpose.

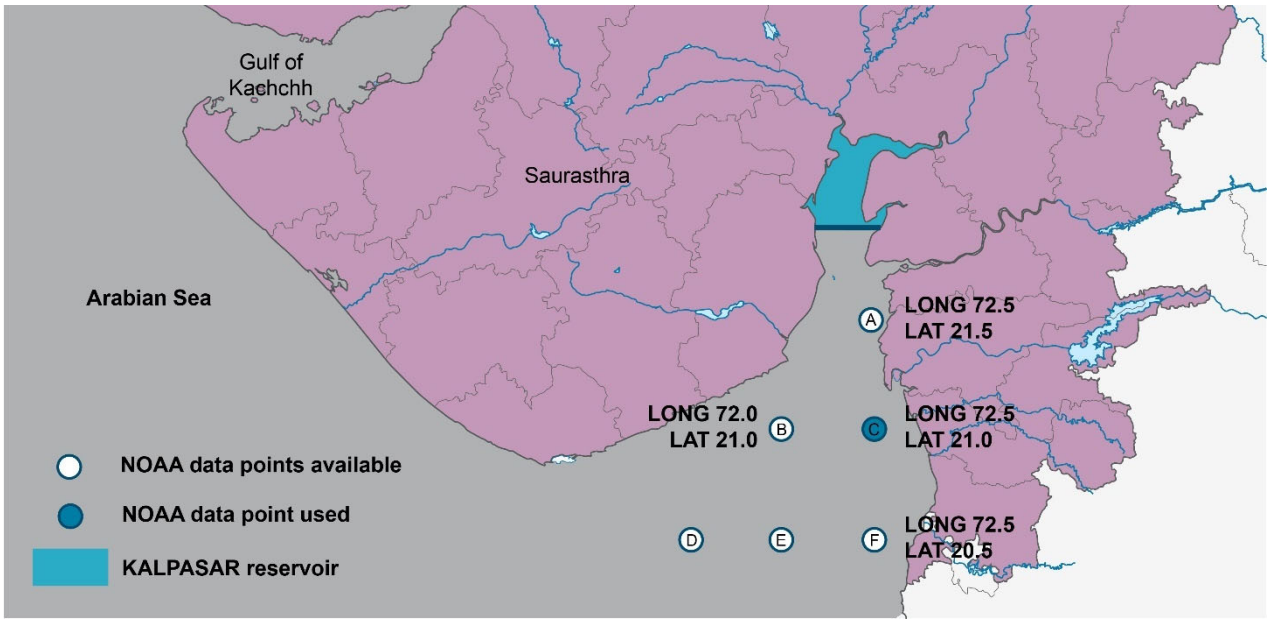


Figure 3.4 NOAA data points in the Gulf of Khambhat and Arabian Sea

The NOAA database at this location covers 31 years of data, from which 196 extreme events are selected by means of peak-over-threshold method to perform an Extreme Value Analysis (EVA). EVA are done for the year-round conditions, conditions outside monsoon season and conditions during monsoon season (Paragraph 3.5). The three EVA are compared in Figure 3.5 and Table 3.5. Details are presented in Appendix A2.

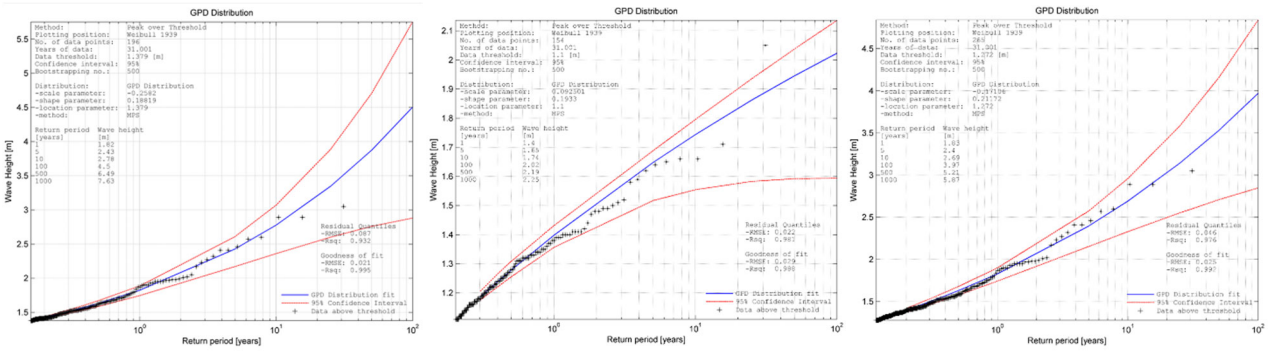


Figure 3.5 Comparison of EVA year-round vs outside monsoon and during monsoon season

Clearly, a wave height of H_s = 1.82 m with a 1/1-year return period should be considered year-round, whereas outside monsoon season, an H_s = 1.40 m is representative with a 1/1-year return period. In case of a 1/100-year return period, H_s = 4.50 m year-round and H_s = 2.02 m outside monsoon season, with H_s = 3.97 m representative during monsoon. For a total construction time exceeding 10 years, a 1/100-year return period is considered for as applicable for the design of the KALPASAR closure dam (~ 0.1 x norm).

Table 3.5 Wave height and return periods (year-round, outside and during monsoon season) at NOAA data point C
** based on average steepness relations of last 3 maximum Hs values for H_s > threshold*

	year round		outside monsoon season		during monsoon season	
return period [yrs]	H_s [m]	T_p* [s]	H_s [m]	T_p* [s]	H_s [m]	T_p* [s]
1/0.2	1.43	5.38			1.40	5.33
1/0.3	1.51	5.54	1.19	4.97	1.50	5.51
1/0.4	1.58	5.66	1.24	5.08	1.57	5.65
1/0.5	1.63	5.76	1.28	5.16	1.63	5.75
1/1	**1.82**	**6.09**	**1.40**	**5.39**	**1.83**	**6.10**
1/5	2.43	7.03	1.65	5.86	2.40	6.98
1/10	2.78	7.51	1.74	6.02	2.69	7.40
1/25	3.34	8.24	1.86	6.23	3.14	7.99
1/50	3.87	8.87	1.95	6.36	3.53	8.47
1/100	**4.50**	**9.57**	**2.02**	**6.49**	**3.97**	**8.98**
1/250	5.53	10.60	2.12	6.64	4.64	9.71
1/500	6.49	11.48	2.19	6.74	5.21	10.29
1/1000	7.63	12.45	2.25	6.84	5.87	10.92

Observations near Dahej are that H_s = 1.8 m for a 1/1-year return period and H_{max} of 3.3 m. An H_s of 4.3 m is representative for a 1/100-year return period at the same site, with H_{max} = 8.0 m. From these observations it is concluded that the wave statistics in Table 3.5 are representative for the wave conditions at the construction site for KALPASAR closure dam. Above is deemed fit-for-purpose for determining the feasibility if the closure dam. A detailed wave study is recommended for the next stage of development. Table 3.6 below shows the wave height per month. As found from Table 3.2, the higher waves occur during monsoon, with winds coming predominantly from the west to south quadrant (Table 3.3).

Table 3.6 Wave height H_{m0} estimates per month at NOAA data point C

	jan	feb	mar	apr	may	jun	jul	aug	sep	oct	nov	dec	year
max	0.91	0.88	0.80	1.04	1.06	1.49	1.68	1.28	1.25	0.79	1.08	0.61	0.91
mean	0.25	0.25	0.26	0.31	0.42	0.49	0.44	0.38	0.29	0.22	0.21	0.23	0.25
stdev	0.09	0.09	0.10	0.13	0.18	0.18	0.15	0.13	0.11	0.08	0.07	0.08	0.09

3.4 Cyclone hazard

Gujarat is highly prone to cyclone risk [6,7]. Researchers found that between 1982 and 2019, there was a significant increase in the frequency, duration, and intensity of cyclonic storms over the Arabian Sea [8]. Specifically, they noted a 52 percent increase in the frequency of cyclonic storms, an 80 percent increase in their duration, and an increase in intensity of about 20 percent in the pre-monsoon period and 40 percent post-monsoon. In addition, researchers documented a tripling of the accumulated cyclone energy in the Arabian Sea.

Climatologically, the frequency of tropical cyclones in the Bay of Bengal is higher relative to that over the Arabian Sea. However, recent years exhibit a greater number of tropical cyclones forming in the Arabian Sea than in the Bay of Bengal. During the study period (1982 – 2019), an increasing trend in the intensity, frequency, and duration of Cyclonic Storms (CS) and Very Severe CS (VSCS) is observed over the Arabian Sea. Recent cyclones in Gujarat occurred in 2009, 2011 and 2015 [9]. All three cyclones occurred in June and were of the category D. Table 3.7 presents the Indian Classification of Cyclonic Disturbances [10].

There is a 52 percent increase in the frequency of CS between 2001 and 2019 in the Arabian Sea, while there is a decrease of 8 percent in the Bay of Bengal. Over the Arabian Sea, increment in CS duration is 80 percent and VSCS is almost threefold compared to 1982 – 2000. Also, lifetime maximum intensity and accumulated cyclone energy have increased over the Arabian Sea implying an increase in the strength of tropical cyclones.

Table 3.7 Indian Classification of Cyclonic Disturbances

type	abbrevation	wind speed [km/h]
Depression	D	31 - 49
Deep Depression	DD	50 - 61
Cyclonic Storm	CS	62 – 88
Severe Cyclonic Storm	SCS	89 - 118
Very Severe Cyclonic Storm	VSCS	119 – 221
Super Cyclonis Storm	SUP CS	> 222

The increase in tropical cyclone duration over the Arabian Sea is prominent during May, June, and October and a decrease over the Bay of Bengal is noted during November. An increase in probability of tropical cyclone intensification should be expected. Significant changes in tropical cyclone tracks are also noted in May, June, and October due to changes in steering currents. Indicative cyclone parameters in relevant

6 India Meteorological Department (2021), “Cyclone warning in India, standard operation procedure”, March 2021, Government of India, Ministry of Earth Sciences.
7 Poulose, J., Rao, A.D. & Dube, S.K. (2020), “Mapping of cyclone induced extreme water levels along Gujarat and Maharashtra coasts: a climate change perspective”, Climate Dynamics 55, 3565–3581.
8 Deshpande, M., Singh, V.K., Ganadhi, M.K. et al (2021), ‘Changing status of tropical cyclones over the north Indian Ocean’, Climate Dynamics 57, 3545–3567.
9 Kantamanen et al (2022), “Appraisal of climate change and cyclone trends in Indian coastal states: a systematic approach towards climate action”, Arabian Journal of Geosciences
10 National Disaster Management Authority (2020), “Cyclone Hazard Prone Districts of India”

districts along the Gulf of Khambhat indicate Probable Maximum Storm Surges (PMSS) between 4.0 m and 5.0 m and wind speeds generally fall within the VSCS category (Table 3.8).

Table 3.8 Cyclone parameters per district adjacent to the Gulf of Khambhat [10]
PMSS: Probable Maximum Storm Surge
PMP: Probable Maximum Precipitation

district	no of severe cyclones	total no of cyclones	wind speeds [km/h]	PMSS [m]	PMP [cm]
Ahmedabad	0	0	160 - 180	4.0	56
Bhavnagar	1	1	160 - 180	4.5	56
Surat	0	0	160 - 170	4.5	88
Navsari	0	1	160 - 170	4.5	88
Valsad	0	0	160 - 170	5.0	104
Bharuch	0	3	160 - 170		72

It should be noted that the different studies below mention different numbers of cyclones that have occurred. However, the different studies also consider wider and more specific areas from the whole of the Arabian Sea area to Gujarat's coastal districts specifically as well as different periods of observation. Therefore, the information is not (necessarily) contradictory.

Finally, [5] considers cyclone risk for the offshore wind farm near Mahuva, some 100 km south along the coast from Bhavnagar. It uses specific events to assess cyclone risk. Of 27 cyclones that have occurred between 1975 and 2015, three have been selected. The cyclone tracks selected are representative for the KALPASAR dam site, and result in maximum significant wave heights greater than 7.0 m at the wind farm location with 1.2 m to 1.6 m. Peak wind speeds and indicative hydraulic conditions at the wind farm site are shown in Table 3.9 and Table 3.10.

Table 3.9 Extreme wind speeds near Mahuva [5]

return period [yrs]	peak wind speed [km/h]	
	3-sec gust	10-min average
10	220	180
50	270	210
100	280	220

Maximum wave height H_{m0} from [5] presented below is typically slightly higher than the significant wave height H_s. It is assumed here that H_{m0} is 5 percent higher, allowing for the estimate of H_s below and comparison with the wave heights presented in Table 3.5 during monsoon season.

Table 3.10 Extreme hydraulic conditions during cyclone event near Mahuva [5]

return period [yrs]	maximum surge height [m]	max H_{mo} [m]	estimated H_s [m]	peak wave period T_p [s]	mean wave direction [°]
10	2.72	8,2	7.8	13,7	186
50	3.26	8,6	8.2	12,9	186
100	3.32	8,6	8.2	12,6	186

Pending further study in the next stage of development of the KALPASAR dam, it is likely that the wave heights at the KALPASAR dam site in case of a cyclone occurring during monsoon will exceed the normal significant wave height during monsoon determined in Paragraph 3.5 by a similar magnitude. The impact of such a coincidence is significant (Table 3.11) and would correspond with wave heights experienced during monsoon exceeding return periods of 1/100 year.

Table 3.11 Indicative comparison of wave conditions during monsoon and in case of coincidence with a cyclone event

return period [yrs]	during monsoon season (Table 3.5) H_s [m]	during cyclone near Mahuva H_s [m]	exceedence of H_s near Mahuva [m]	estimated H_s during monsoon and cyclone coincidence at site H_s [m]	estimated equivalent return period during monsoon
1/1	1.83				
1/10	2.69	7.8	+1.2	3.9	1/100
1/50	3.53	8.2	+1.6	5.1	1/500
1/100	3.97	8.2	+1.6	5.6	1/800
1/250	4.64				
1/500	5.21				
1/1000	5.87				

Clearly, there is an increasing cyclone risk in the Gulf of Khambhat. Cyclones typically occur during or around monsoon season and return periods are such that cyclone risk cannot be ignored given the long duration of construction of the KALPASAR dam (10 – 15 years). The impact is only estimated, but severe damage during construction should be anticipated. It is noted that the actual impact is strongly dependent on spatial variation of the cyclone tracks. Above estimates are determined from historical cyclone tracks in or near the Gulf of Khambhat. It is strongly advised to study potential impact of cyclone hazard adjacent to site specific wave studies when progressing onto the next stage of development of KALPASAR dam.

3.5 Monsoon characteristics

Monsoon season typically starts in June and ends in September. The peak in rainfall occurs in July (Figure 3.6) although a trend is found indicating that the peak in rainfall shifts towards the second half of the monsoon season in the last decades [11]. Table 3.12 shows monsoon statistical characteristics based on 1871 – 2011 rainfall data for Gujarat, which clearly shows rainfall between 80 mm and 120 mm per day during the monsoon season attributing to 95 percent of the total annual rainfall in the state. Pre-monsoon and post-monsoon, rainfall is typically substantially less (respectively 12 and 26 mm per day). However, it should be noted that cyclones occur adjacent to monsoon season and may result in similar amounts of rainfall and unworkable conditions in May and October as well.

Table 3.12 Monsoon statistical characteristics [11]

	rainfall [mm] (1871 - 2011)			
month	mean	standard deviation	CV	contribution to annual
jan	1.6	4.0	244.4	0.2
feb	2.1	6.4	298.1	0.3
mar	0.9	4.6	498.1	0.1
apr	1.2	5.0	422.3	0.1

11 Chinchorkar et al (2020), “Temporal Variability and Trend in Monthly, Seasonal and Annual Rainfall in Twentieth Century over Gujarat Region”.

month	rainfall [mm] (1871 - 2011) mean	standard deviation	CV	contribution to annual
may	6.2	14.6	235.0	0.7
jun	117.3	82.8	70.6	14.0
jul	329.2	191.2	58.1	39.2
aug	222.7	119.9	53.8	26.5
sep	131.8	108.5	82.3	15.7
oct	18.4	31.9	173.2	2.2
nov	6.3	15.1	241.5	0.7
dec	1.5	3.6	244.2	0.2
annual	839.2	289.6	34.5	100.0
pre-monsoon	12.1	18.3	150.9	1.4
post-monsoon	26.1	34.3	131.2	3.1

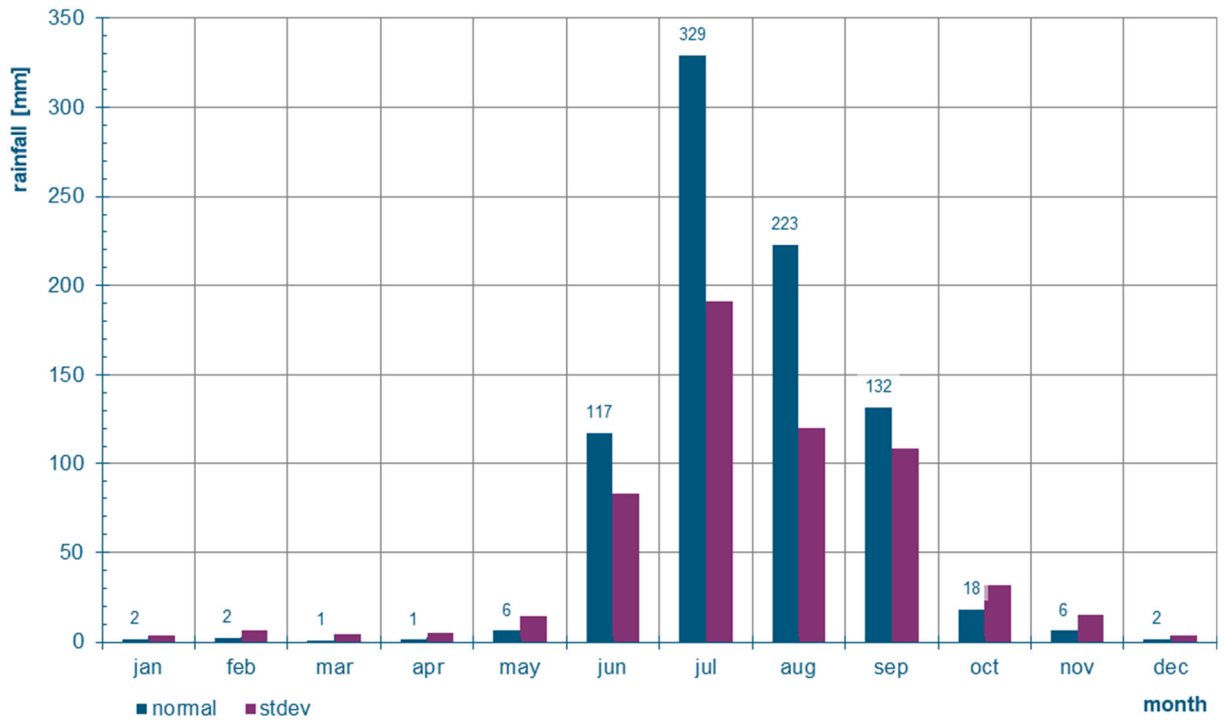


Figure 3.6 Monsoon rainfall (average and standard deviation | 1871 – 2011 observations)

3.6 Summary of key hydraulic conditions

A summary of seasonal variation of key hydraulic conditions during construction of the KALPASAR closure dam is presented in Table 3.13 below.

Table 3.13 Monthly variation of key hydraulic conditions (summary)

month	average wind speed [m/s]	wave height max H_{m0} [m]	rainfall [mm]	cyclone risk [-]	estimated max wave height during monsoon and cyclone coincidence [m]	storm surge [m]
jan	5.55	0.91	1.6			
feb	4.84	0.88	2.1			
mar	4.40	0.80	0.9			
apr	4.61	1.04	1.2			
may	6.07	1.06	6.2	yes, pending further research in the next stage of development	3.9 m – 5.6 m	yes, pending further research in the next stage of development
jun	7.24	1.49	117.3			
jul	7.69	1.68	329.2			
aug	6.67	1.28	222.7			
sep	5.22	1.25	131.8			
oct	3.43	0.79	18.4			
nov	4.45	1.08	6.3			
dec	5.24	0.61	1.5			

3.7 Soil conditions

For relevant soil conditions along the KALPASAR closure dam corridor, reference is made to the [12].From this report it is concluded that the soil conditions in 3 zones (Zone 1, 6 and 7) are predominantly clayey in nature and in 4 zones (Zones 2, 3, 4 and 5) are in sandy in nature. The presence of layers of soft clay and very loose to loose silty sand layers in the top soil strata emphasizes that ground improvement is necessary prior to construction of the KALPASAR closure dam to mitigate the ground settlement and improve the bearing capacity, similar to what was found along the southern alignment [1].

At present, the feasibility of the KALPASAR closure dam is assessed and here it is assumed that such ground improvement works are adequately and accurately implemented prior to construction of the KALPASAR closure dam. Therefore, the feasibility of the closure dam will not be (adversely) affected by the ground improvement works. Note that ground improvement may take several years to implement and has to be accounted for in the feasibility of the overall project.

12 ITTM, “Geotechnical Aspects and Dam Design”.

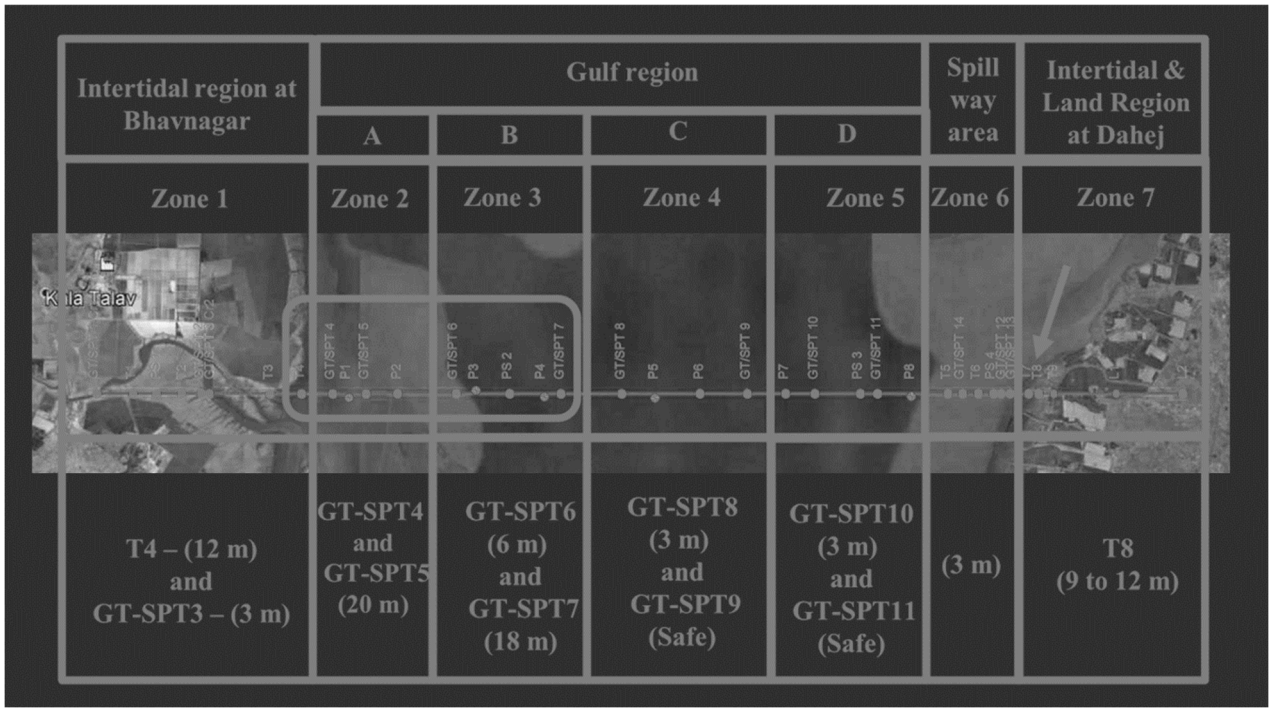


Figure 3.7 Locations of boreholes and zones [12]

4 DESIGN BASIS

A design basis is determined from analysis of the site conditions and monthly variations in order to determine the (average) length of construction season and subsequently the hydraulic design conditions to account for during and outside construction season (survivability). This concerns primarily the gradual closure components. Specific attention is given to hydraulic conditions during (sudden) closure and the likelihood of occurrence of extreme events in general.

4.1 Main stages of the closure dam construction

Construction and preparation of the final closure starts once the maximum (horizontal and vertical) gradual closure is established (Figure 4.1). At this point, flow velocities would exceed the stability of the rock used in the gradual closure, at least from the perspective of construction economics. The vertical closure dam erected to MSL -12.0 m approximately is the established sill which then becomes the foundation for the closure elements.

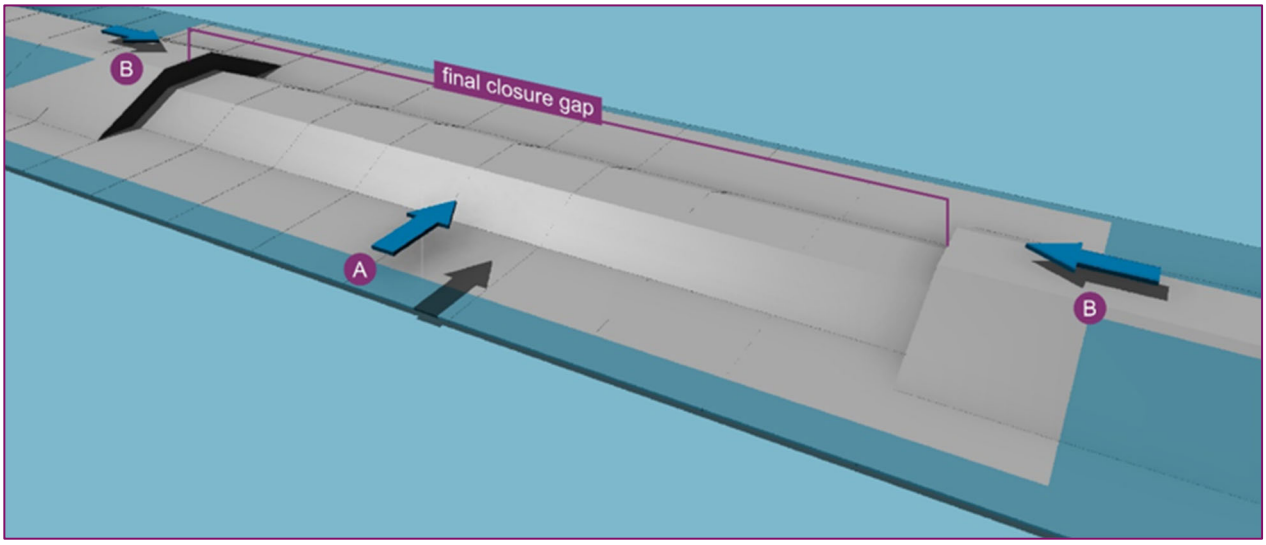


Figure 4.1 Construction process - *Gradual vertical closure (A); gradual horizontal closure (B); final closure gap for sudden closure*

With the final closure gap spanning more than 10 km, the number of closure elements is significant. Figure 4.2 illustrates the construction process from the moment a closure element arrives on site.

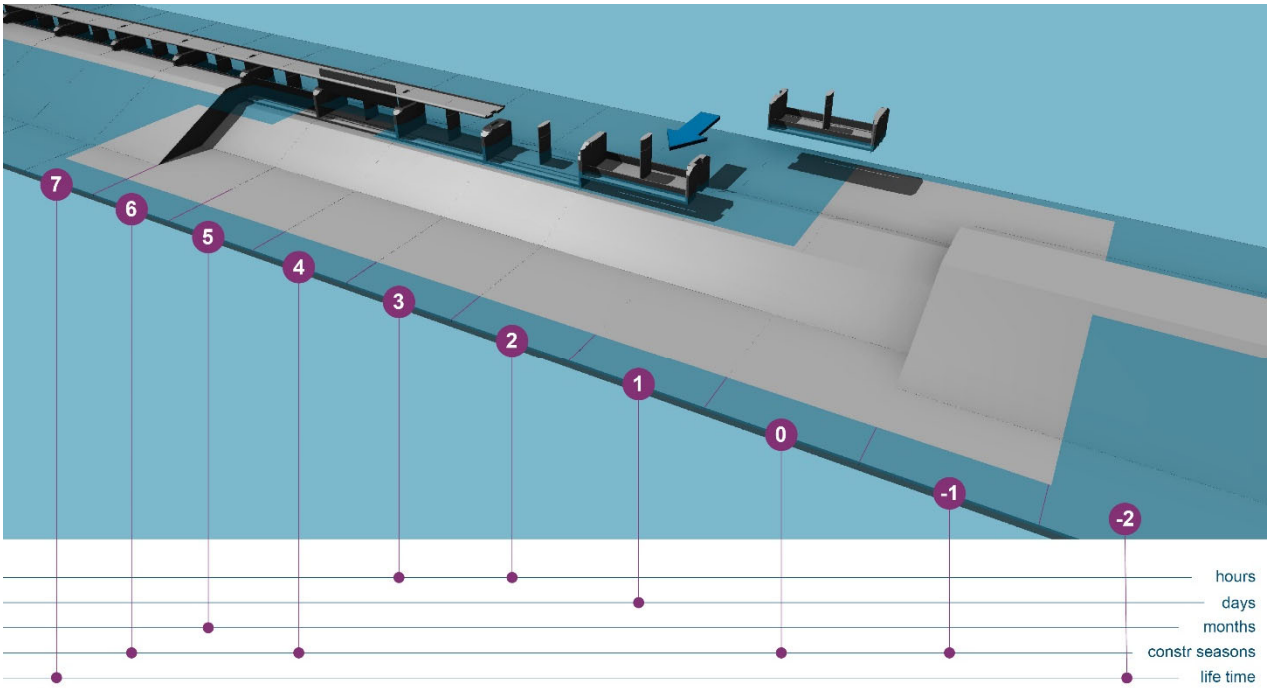


Figure 4.2 Construction of the final closure gap

The process and preparation for the sudden closure starts once a sill is established (0 in Figure 4.2). The sill is the final stage of the gradual vertical closure component. The rock sill elevated to MSL -12,0 m with a crest of at least 50 m wide. The sill crest must be prepared for accurate placement of the closure elements is possible. Pending further design, an interface layer may be needed to level out the sill crest. Levelling out is assumed to be done by placement of block mattresses of specific height. The block matrasses are aligned with the base slab of the closure elements. While this requires additional handling, the installation of such an interface layer does not directly affect failure risk of the actual sudden closure: Levelling out the sill in this manner concerns proven technology and accurate placement of the closure element therefore feasible. Importantly, the (levelled) sill may require a survivability of several construction seasons before the closure elements are placed on top of it.

Depending on optimisation of production and logistics, closure elements are then transported to a waiting area (1 in Figure 4.2), before they are finally (individually) manoeuvred to their final position (2 in Figure 4.2) and ballasted for the element to submerge and connect to the foundation (3 in Figure 4.2). Closure elements may be parked in a waiting area for several days, the positioning and ballasting is a matter of several hours.

Note that suitable working conditions must be found. With a substantial amount of closure elements to be placed, an equal number of operational windows must be identified across one or more construction seasons. Working conditions and subsequent time needed to install the closure elements are discussed in Paragraph 8.2). This then determines the logistics and off-site preparation of closure elements to minimise the installation time of the closure elements and to so limit exposure duration of unfinished construction components (Paragraph 8.2). The need for overall minimising of installation time addresses is limited to max 6 years in order to allow for timely delivery of the KALPASAR dam project as a whole (Chapter 2).

4.1.1 Sudden closure procedure

The closure element is then left for a longer period of up to several construction seasons before the actual sudden closure is done (4 in Figure 4.2). During this period, if the closure element was not pre-fitted with a bridge deck, a bridge deck can be installed, either working forward from the sides (traditional bridge building method) or from pontoons and cranes. Again, like the interface layer between the sill and closure element, this does not affect the failure risk of the sudden closure, rather it is a logistic optimisation.

Once all closure elements are placed in the closure gap and adequately interfaced with the horizontal closure components at the land side(s), preparations for the sudden closure may commence. At this point, first all gates have to be placed in the closure elements (5 in Figure 4.2). It is assumed that this will take one or two construction seasons at least before all gates are installed and prepared for deployment.

Rather than to pre-install the gates, retrofitting is preferred for now, because (i) the risk of damage during transport and positioning, (ii) for the risk of damage to the gates during the preparatory stage before deployment of several construction seasons, and (iii) assumed optimised logistics since the installed bridge deck can be used during preparations. In doing so the risk of damage is reduced to a practical minimum. An exposure time of the prepared gates to an order of months before deployment seems feasible pending further study. Once all gates are installed and prepared for deployment, all gates must be lowered simultaneously (6 in Figure 4.2).

Pending further optimisation, smaller gates can potentially be installed for deployment from the bridge deck, whereas larger ones are likely to require installation from floating cranes, making production dependent on tidal working conditions which may result in a production lower rate. Either can be considered, e.g., slow installation and less gates to simultaneously close, versus faster installation and more gates required to simultaneously close (Paragraph 8.2).

4.1.2 Completion of the (closure) dam

This procedure of simultaneous lowering of the gates in the final closure gap is evaluated in a quantitative risk analysis (see Paragraph 8.4). The situation where all gates are closed may also last one or more construction seasons before the final (seaward) form is achieved (7 in Figure 4.2). Once this is done, the gates are effectively obsolete. The gates can potentially also be upgraded to become (part of) a sea wall or they are retrieved or left in place and buried underneath the final construction. Either is possible and pending cost-optimisation of the KALPASAR dam as a whole.

4.2 Construction season window per stage / activity

From the previous chapter it follows that the monsoon season typically lasts from June to September, resulting in unworkable conditions that last at least three months and at the most four months. For construction modelling, three months of unworkable conditions are assumed. Thereby it is implicitly assumed that some construction activities that are less subject to site conditions can still progress in the lead-up to shut down for monsoon season or in preparation of the next construction season in the month of September, even though working conditions at sea cannot be resumed yet. This means that for actual construction of the KALPASAR closure dam (on-site) is limited to eight months per year.

In total, for the feasibility assessment of the KALPASAR closure dam, 5 main construction stages and activities are distinguished and listed in Table 4.1. In the same table, for the relevant of key hydraulic design parameters / risks per month are presented to allow for item-specific hydraulic design basis and subsequently risk-assessment in the main report. Note that the flow velocities and change in water level during the gradual closure is accounted for via the hydraulic and construction model as progress is made.

Table 4.1 Relevance of key hydraulic design parameters / risks per month, per construction stage and activity

	average wind speed [m/s]	wave height [m]	rainfall [mm]	cyclone risk [-]	estimated max wave height during monsoon and cyclone	storm surge [m]	construction of gradual closure	survivability of during monsoon	deployment of closure components	sudden closure of closure components	survivability post-closure	
jan	5.55	0.91	1.6									
feb	4.84	0.88	2.1									
mar	4.40	0.80	0.9									
apr	4.61	1.04	1.2									
may	6.07	1.06	6.2	yes	3.9 m – 5.6 m	yes						
jun	7.24	1.49	117.3									
jul	7.69	1.68	329.2									
aug	6.67	1.28	222.7									
sep	5.22	1.25	131.8									
oct	3.43	0.79	18.4									
nov	4.45	1.08	6.3									
dec	5.24	0.61	1.5									

In the paragraphs hereafter, the different stages of construction / construction activities are discussed more specifically.

4.3 Conditions during gradual closure

During construction of the gradual closure components, hydraulic conditions change and in terms of suitability for construction, unfavourably. The undisturbed tidal signal is presented in Paragraph 3.1. Once the vertical closure component is finalised (elevation MSL -12,0 m) the tidal signal will be disturbed, and flow velocities will have increased as opposed to the undisturbed situation in Figure 3.3 to those shown in Figure 4.3. Once horizontal closure progresses, conditions will again gradually change: Figure 4.4 shows the computed neap, average and spring tide and associated profile average velocities in case of a 10 km final closure gap. Note that once the KALPASAR closure dam is fully completed, a larger tide in front of the dam should be anticipated [2]. This effect of an amplified tide is not found from the indicative one-dimensional simulations but as construction progresses, this may occur locally already. Such impact should be analysed in the next stage of development.

Hence, the design tidal conditions at the start deployment of the concrete closure elements and gradual horizontal closure is the tidal signal at 5 km closure shown in Figure 4.3, e.g., with only the logistics and construction hubs constructed. During the sudden closure stage (e.g., when the horizontal closure has been completed adjacent to the concrete closure elements sections) the tide as shown in Figure 4.4 is representative.

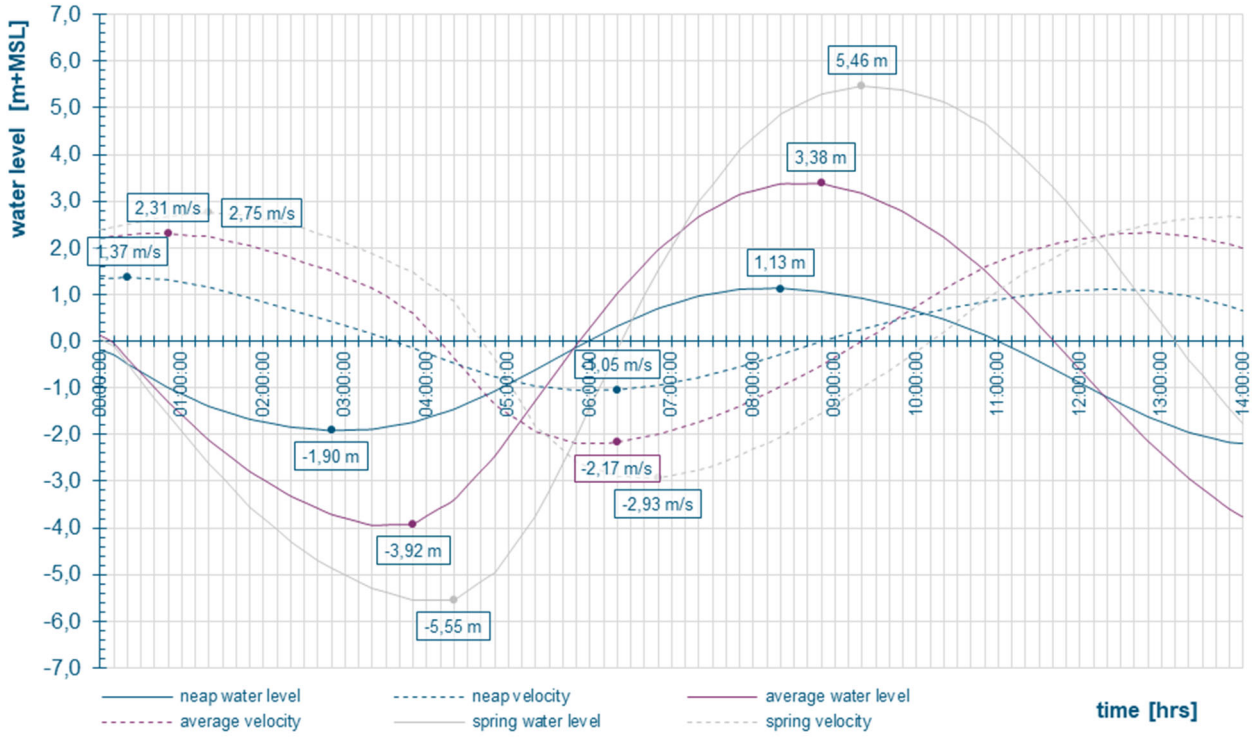


Figure 4.3 Computed neap, average and spring tide and associated profile average velocity near dam location prior to logistics and construction hubs reclamation and vertical closure component completed up to MSL -12.0 m, representative for start of concrete element deployment.

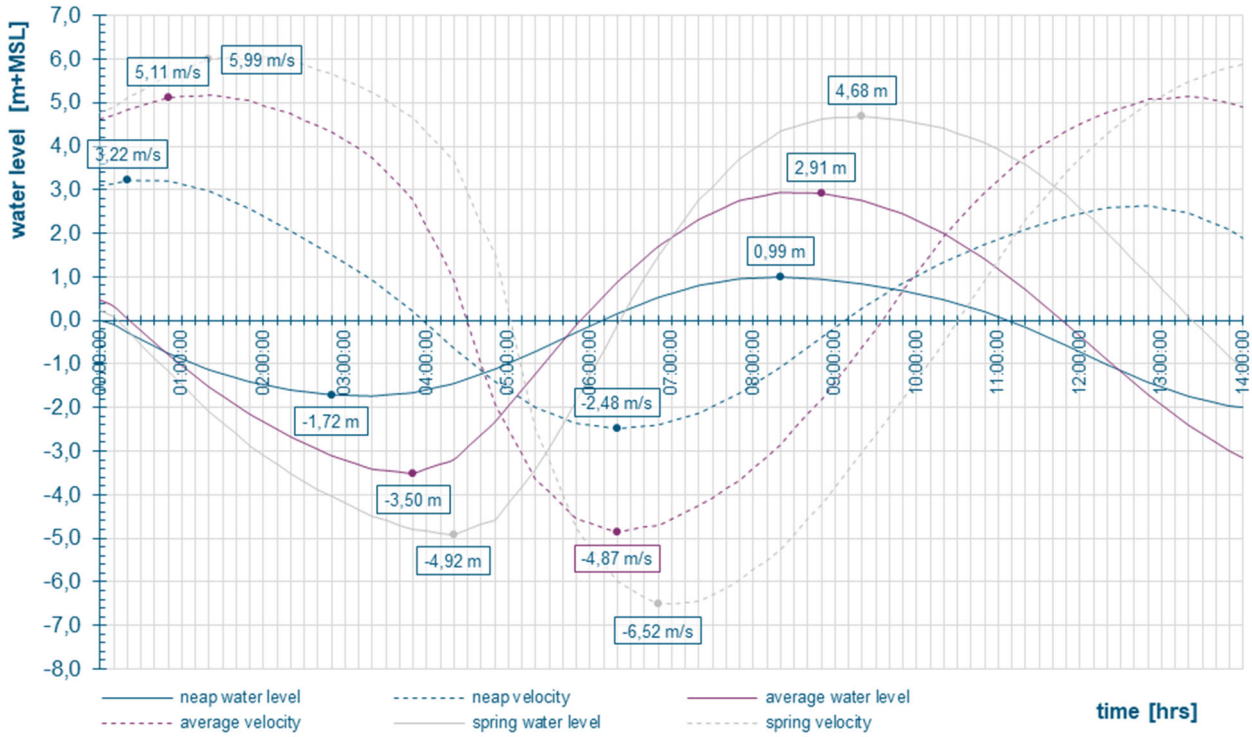


Figure 4.4 Computed neap, average and spring tide and associated profile average velocity near dam location with logistics and construction hubs reclaimed (total horizontal closure progress 20 km) and vertical closure component completed up to MSL -12.0 m.

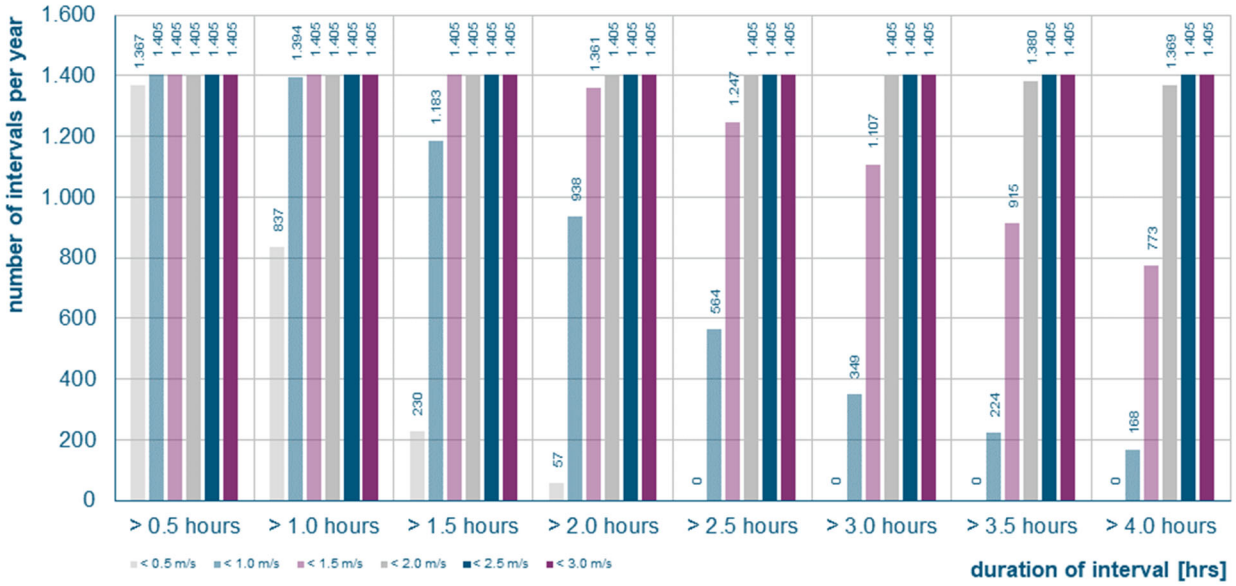
The increase in flow velocities is accounted for in the construction model (see Paragraph 7.1) and once flow velocities exceed a practical standard rock grading stability, the gradual closure has reached its maximum extent and the remaining gap has to be closed by means of the sudden closure component. The impact of the horizontal closure component on the sudden closure component hydraulic conditions is discussed in Paragraph 4.5.

4.4 Conditions during deployment of open closure elements

The sudden closure consists of, (i) the deployment of the open concrete closure elements and preparatory work, and (ii) the actual sudden closure of the deployed open elements all at once. Subsequently, (hydraulic) design conditions between taking the concrete closure element from the parking area and positioning it above its final place in the closure gap and during the actual lowering are also different.

It is assumed that a purpose-built lifting vessel with DP2 or DP3 [13] can take a concrete closure element from the parking area and accurately hold position in flow velocities up to 2.0 m/s or 2.5 m/s at least. The actual lowering onto the vertical closure component will then be done when flow velocities are at their lowest around slack tide with flow velocities lower than 1.0 m/s. It is further assumed that at least a four-hour window is needed for positioning to include some redundancy in case lowering the concrete closure element somehow fails or if adjustments are needed.

Within the four hours, a briefer window for the lowering will be available. In that case, in the situation where the vertical closure has reached MSL -12.0 m and without any horizontal closure (e.g., also without logistics and construction hubs reclaimed), there are 1.405 windows per year during which flow velocities are below 2.5 m/s and 1.389 during which flow velocities are below 2.0 m/s (Figure 4.5).



13 According to IMO guidelines, DP systems are rated DP1, DP2, or DP3. The levels build on each other, with each succeeding level having the abilities of the previous level. DP1 systems are the most basic, with the ability to keep their position in automatic mode. DP2 fulfils DP1 requirements but can also keep station with the failure of an active component. The redundant system must provide the ability to keep station until work can be safely stopped, and the transfer of operations must be automatic. DP3 takes DP2 one step further with the ability to continue operation with the failure of an active or static component. DP1 vessels are not very common, with DP2 and DP3 becoming more common due to the additional level of safety they provide.

Figure 4.5 Number of intervals per year at 0 km horizontal closure progress
0 km of horizontal closure is representative for a construction scenario without logistics and construction hubs reclaimed.

In the case wherein construction of the horizontal closure has progressed by 15 km, it is readily seen that the number of windows for deployment of the concrete closure elements is significantly reduced (Figure 4.7). The number of windows lower than 2.5 m/s is reduced from 1.405 to 1.352 and the number of windows lower than 2.0 m/s is reduced from 1.389 to 1.110 per year. Once horizontal closure has progressed to 20 km the number of four-hour windows is further reduced (Figure 4.7). At this point it is also clear that the window wherein the flow velocities are below 0.5 m/s dramatically reduces also, meaning that less time would be available to perform the actual sudden closure (see Paragraph 4.5 hereafter) and that the steepness of the tidal signal has increased.

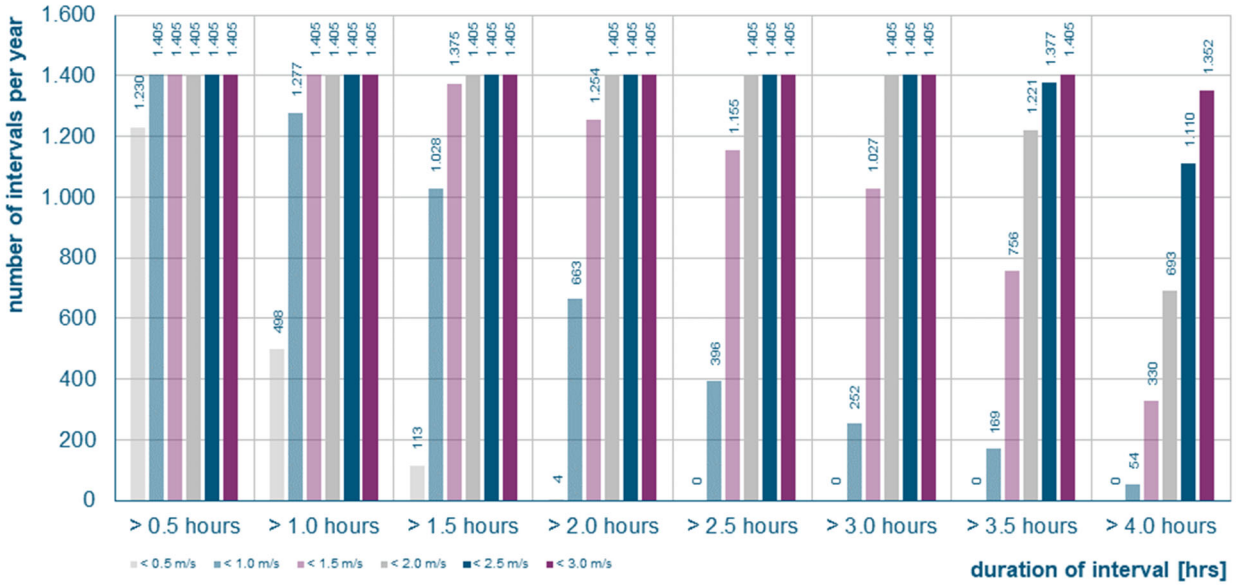


Figure 4.6 Number of intervals per year at 15 km horizontal closure
15 km of horizontal closure is representative for a construction scenario with 5 km logistics and construction hubs reclaimed and 10 km of horizontal closure.

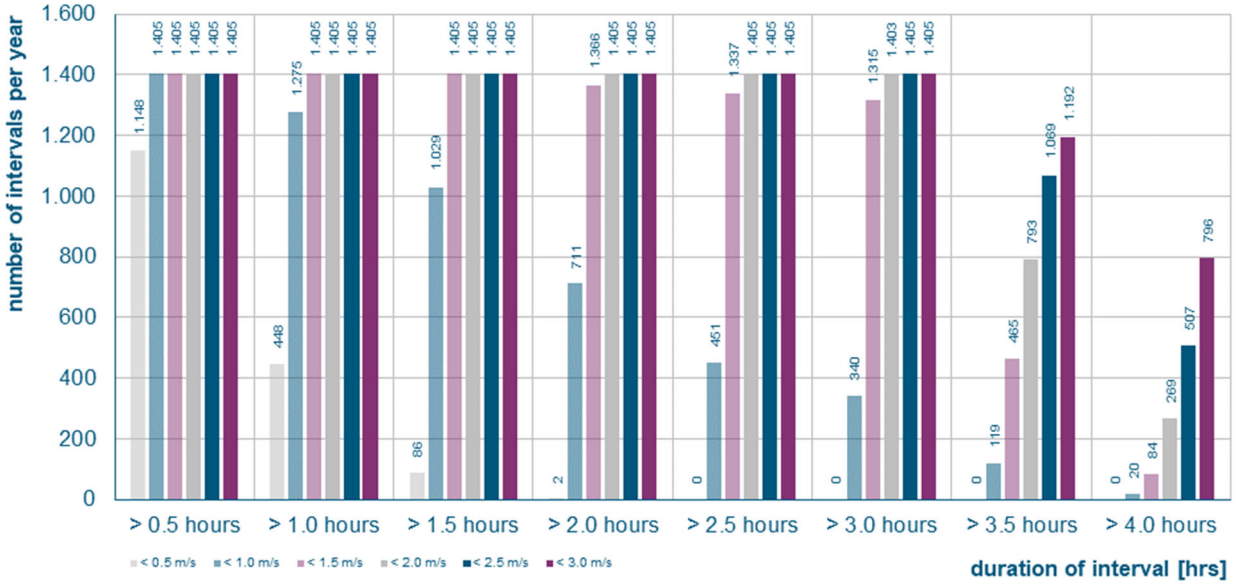


Figure 4.7 Number of intervals per year at 20 km horizontal closure
20 km of horizontal closure is representative for a construction scenario with 5 km logistics and construction hubs reclaimed and 15 km of horizontal closure.

Presenting the availability of four-hour windows as a function of horizontal closure progress (Figure 4.8), it becomes clear that progress made in horizontal closure adversely affects not only the (hydraulic) working conditions on the remaining (sudden) closure component, but also the time available for construction or rather construction conditions and risks.

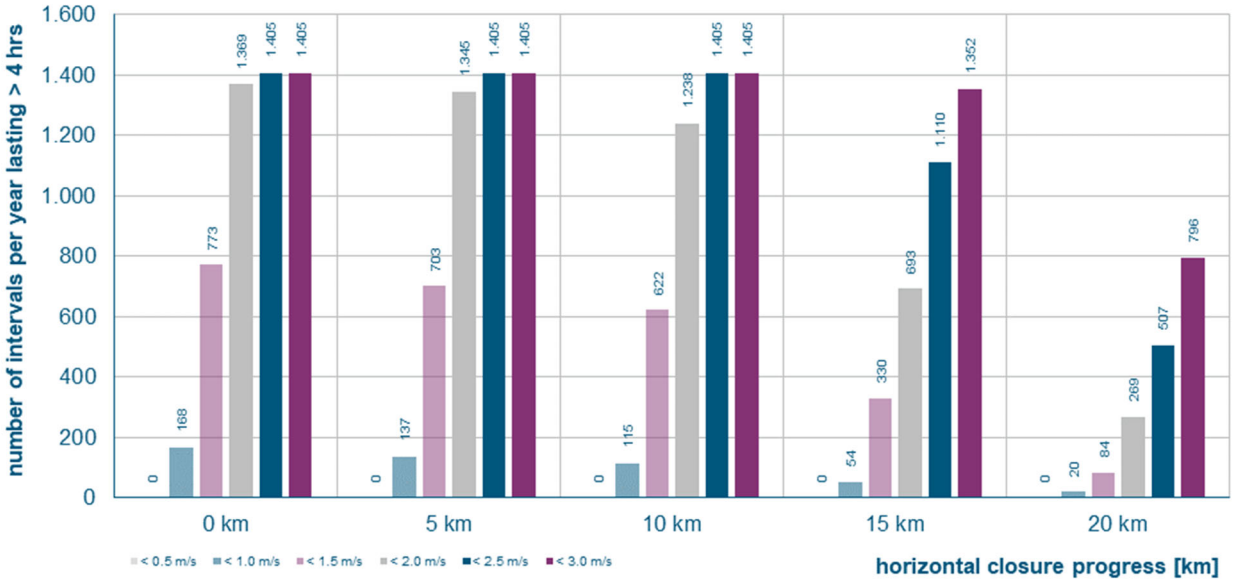


Figure 4.8 Comparison of number of intervals of 4 hrs as a function of horizontal closure progress
0 km of horizontal closure is representative for a construction scenario without logistics and construction hubs reclaimed; 5 km of horizontal closure is representative for a construction scenario with logistics and construction hubs reclaimed.

This becomes even more apparent as the number of windows do not account for monsoon season and include windows during the night. Further, with a semi-diurnal tide, it is assumed that the two high tides per day are not used for deployment either as this will, (i) require more time for lowering the concrete elements onto the bed because of the greater depth, and (ii) inherently involve more risk. Working at night is at present also not considered for deployment operations. Therefore, the number of windows must by reduced with 33 percent to account for a four-month monsoon period and 75 percent for daytime operations and lowering during low tide. This results in the number of windows presented in Figure 4.9, again as a function of horizontal closure progress.

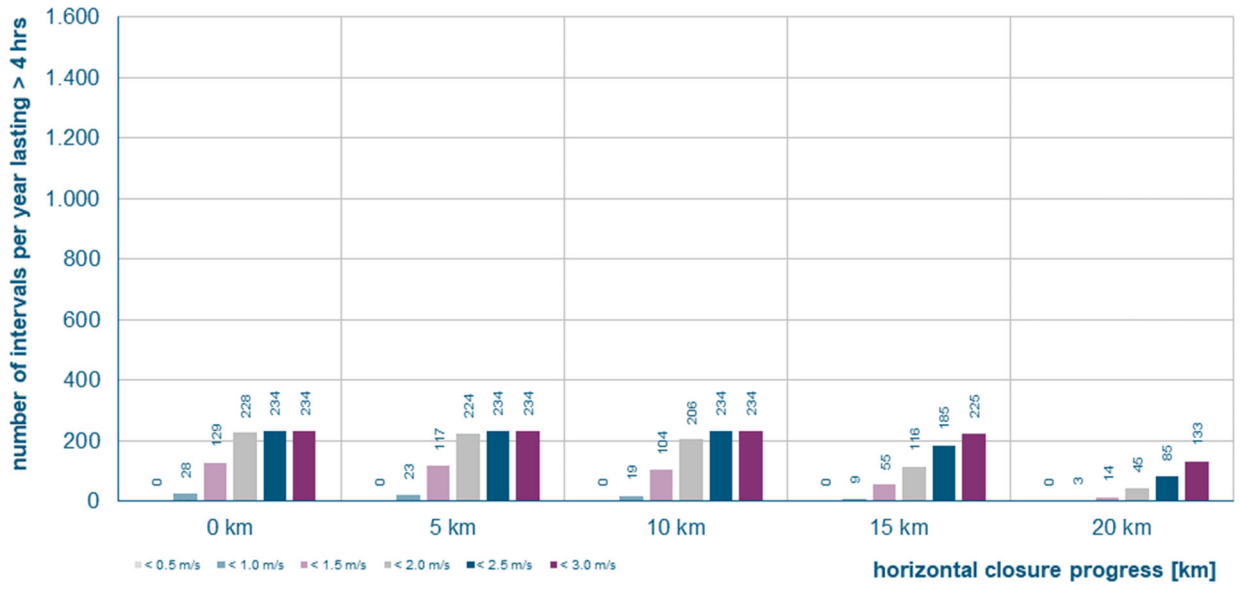


Figure 4.9 Comparison of number of intervals of 4 hrs as a function of horizontal closure progress corrected
Assuming 8 months per year available for (offshore) construction due to monsoon (67 percent) and working during daylight only (25 percent for a semi-diurnal tide) or 18 percent of the total intervals in Figure 4.8. 0 km of horizontal closure is representative for a construction scenario without logistics and construction hubs reclaimed; 5 km of horizontal closure is representative for a construction scenario with logistics and construction hubs reclaimed.

The analysis demonstrates that in case horizontal closure has progressed by 15 km (including 5 km of logistics and construction hubs), the number of actual windows of at least four hours below 2.5 m/s is 185 per year and that the actual windows below 2.0 m/s is 116. At 20 km of horizontal closure progress (including 5 km of logistics and construction hubs), the number of windows below 2.5 m/s is reduced to 85 per year and the number of windows below 2.0 m/s to 45. A drop-off point in terms of construction windows clearly exists somewhere between 10 km and 15 km progress of horizontal closure.

For a closure of this size and the number of concrete closure elements needed (150 in case of 100 m elements and 15 km closure remaining), the time needed for deployment of all closure elements would subsequently take some 2 years, possibly causing the total target construction time for the KALPASAR closure dam to be exceeded.

Particularly for the smaller final closure gaps, this dictates that deployment of the open concrete closure elements starts prior to any progress made with the horizontal closure (except for the logistic and construction hubs) as soon as the vertical gradual closure component allows and to begin the horizontal closure only when all concrete closure elements are their final position. By doing so the number of four-hour windows is 234 (for velocities < 2.5 m/s) and 224 (for velocities < 2.0 m/s). This would roughly mean an option for deployment of an element exist every day outside monsoon season.

Regardless of the above, a long period of deploying the concrete closure elements would be necessary still: The duration of deployment can be further optimised by (i) working in parallel with the vertical closure component construction and (ii) by considering two dedicated lifting vessels to increase capacity, by which at least 468 elements can be deployed per year, notwithstanding momentary weather conditions. For the present feasibility study for the KALPASAR closure dam, it is assumed that at least 500 elements can be deployed per year with certainty, if two vessels are used and if work is carried out in parallel to the vertical closure works.

In turn this also requires that construction of the concrete closure elements begins as soon as soil improvement and work on the logistics and construction hubs begins, to ensure enough are available for continuous deployment. Doing otherwise will mean that the KALPASAR closure dam construction and so the construction of the KALPASAR dam as a whole is not feasible within the desired timeframe of 10 to 15 years.

Depending on the configuration of the concrete closure element at the time of deployment (e.g., with or without a pre-fitted bridge deck, steel gates and mechanical and electrical equipment), the same windows apply for retrofitting the gates, assuming that a bridge deck is fitted working forward from land and that the gates are fitted from a lifting vessel.

It should be noted that within the present scope, the spillway is constructed on the eastern bank of the Gulf of Khambhat outside the 30 km closure dam. Given the difference in number of available construction windows and tidal conditions at various extents of horizontal closure, it is strongly advised to reconsider its location as it would allow to control and optimise the construction windows and tidal conditions: for instance, 10 km of sudden closure length versus 15 km of closure length results in almost twice the number of windows for deployment of concrete closure elements below 2.0 m/s. The same increase in working windows is achieved comparing 20 km of horizontal closure length to 15 km of closure horizontal closure length (Figure 4.9). With a spillway width of roughly 2.0 km to 2.5 km, repositioning it to within the KALPASAR closure dam's extent allows for significant optimisation of the working windows.

INTERIM CONCLUSIONS – I
a) Beyond horizontal (gradual) closure progress of 10 km, there is a drop-off point which limits deployment operation windows per year. While from a rock stability point of view, further progress may still be possible, the overall time needed to deploy all open concrete closure elements increases as less elements can be deployed, even at increased construction / deployment capacity. It is therefore assumed that deployment of the concrete closure elements is done after completion of (a part of) the vertical closure component and before the horizontal closure begins, to have maximum opportunity for swift deployment of the large number of concrete closure elements.
b) The deployment rate of open concrete closure elements is at least 500 per year if they are deployed prior to horizontal gradual closure with two dedicated lifting vessels. This is a conservative estimate taking into account unworkable weather conditions.

4.5 Conditions during sudden closure of deployed closure elements

The previous paragraph concerned the deployment of the concrete closure elements in open condition. The sudden closure occurs once also the horizontal gradual closure component is completed. The number of windows to perform the sudden closure of lowering the gates in the open elements all at once is also affected by the extent of horizontal closure: the more progress has been made the less windows of conditions where flow velocities are below 1.0 m/s for a four-hour period. For example, the number of four-hour intervals with a flow velocity below 1.0 m/s through the concrete closure elements in open condition is only 3 times a year with 20 km of horizontal closure progress, as opposed to 19 windows that exist when 10 km of horizontal closure progress has been made (including 5 km of logistics and construction hubs reclaimed).

For reasons of survivability of the sudden closure section, which will need protection ahead of the next monsoon season, the number of windows for performing the sudden closure is essential to have enough windows soon after the monsoon season to ensure a maximum amount of time is available to work on these protection works (see Paragraph 8.2), in case monsoon seasons last longer than expected or in case of unsuitable weather conditions occurring (see Paragraph 3.6). From this point of view, it is more favourable to have a smaller horizontal closure section and a larger sudden closure section than the other way around. With 19 windows at 10 km of horizontal closure progress, it is assumed that some three opportunities are available in the first month of monsoon leaving seven months for work on the 20 km long protection works; with 15 km of horizontal progress this would mean having 1 or 1.5 windows in the first month, which if they cannot be used means that less time is available to protect the 15 km of protection works.

For illustrative purposes, Figure 4.10 below shows the number of windows lasting two hours as a function of horizontal closure process:

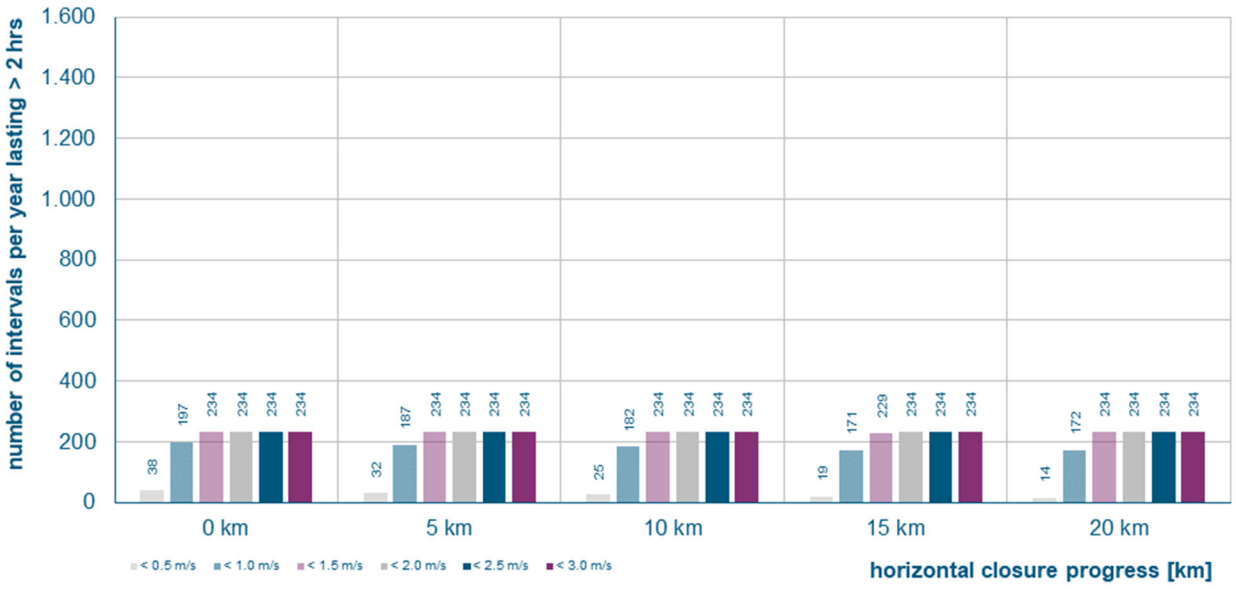


Figure 4.10 Comparison of number of intervals of 2 hrs as a function of horizontal closure progress corrected
Assuming 8 months per year available for (offshore) construction due to monsoon (67 percent) and working during daylight only (25 percent for a semi-diurnal tide) or 18 percent of the total intervals in Figure 4.8. 0 km of horizontal closure is representative for a construction scenario without logistics and construction hubs reclaimed; 5 km of horizontal closure is representative for a construction scenario with logistics and construction hubs reclaimed.

A two-hour window is used assuming it will give one hour before and after the turning of tide where velocities are lower than 0.5 m/s. Thereby allowing for some margin and conditions where (limited) intervention can be done if the process stalls. If the horizontal closure component is 20 km, there are in fact 14 windows in the 8 months between monsoons, so roughly two per month. As it will be necessary to protect the closed concrete closure elements for survivability before the next monsoon at least, the more windows there are, the more certain it is that enough time is available for doing so. Therefore, in principle it matters whether there are 2 windows in the first month after monsoon or 4 (in case of 5 km of horizontal closure), as also weather conditions will determine whether the sudden closure will be triggered.

Similar to improving conditions during deployment of the concrete closure elements, relocating the spillway and keeping it open during the sudden closure process will improve conditions as it will result in a longer window of lower flow velocities as the tidal signal becomes less steep. This may potentially add one or two windows for sudden closure, pending further evaluation in the next stage of development.

INTERIM CONCLUSIONS – II
The maximum horizontal closure component is determined by the rock stability under the increasing flow velocities as progress is made. The maximum horizontal closure component lies between 15 km to 20 km of horizontal gradual closure. However, such a size of horizontal closure means that the number of opportunities for performing the sudden closure soon after monsoon season ends is limited. The size of horizontal closure is likely to be limited by the need to have ample options to perform the sudden closure rather than by hydraulic conditions. In short: from a sudden closure perspective, it is more favourable to have a shorter horizontal closure component and a longer sudden closure component, notwithstanding the failure risk of a comparatively longer sudden closure component.

Hydraulic conditions depend on the preferred length of horizontal closure and varies between the conditions in Figure 4.3 and Figure 4.4.

4.6 Conditions after sudden closure

After closure of the final closure gap, rather than waves passing through the open structure as has been the case prior, wave will reflect off the vertical sea wall thus erected. This implies an additional load case to account for in the design of the concrete closure elements as well as the adjoining bed protection / rock bund.

4.7 (Rock) material sourcing

It has the preference of the Client to source material from Indian quarries. Table 4.2 [14] below indicates the quantities of different gradings available from quarries west and east of the Gulf of Khambhat. In total there are 44 quarries. The sourcing from these quarries requires road transport of large quantities of rock. The quantity stated in column two is understood to be an estimate of the total amounts of rock needed for the KALPASAR dam, rather than what is needed for only the KALPASAR closure dam. From the total volume of rock available (267.40 mln m^3), it is understood that 167.89 mln m^3 is needed for KALPASAR dam. This entails 63 percent of the total volume available from the quarries. While apparently sufficient, this is likely to drive up prices to deliver a constant supply of rock of, say 17 mln m^3 of rock per year with 10 years of construction time and the need to supply other projects at the same time as well. A price risk clearly exists. Additional sources may be considered to mitigate such risks or to optimise logistics.

Table 4.2 Material availability and demand as provided by NCCR

grading	quantity [mln m^3]	western quarries [mln m^3]	eastern quarries [mln m^3]	total available [mln m^3]
8 – 10 t	3	0.95	19.22	20.17
4 – 6 t	3	0.56	9.17	9.73
2 – 4 t	10	1.39	15.83	17.22
1 – 2 t	50	5.98	12.03	18.02
0.5 – 1 t	12	12.60	10.01	22.62
300 – 500 kg	35	14.15	13.31	27.46
200 – 300 kg	2	3.77	4.04	7.81
10 – 500 kg	40	-	-	-
1 – 10 kg	3	51.42	38.38	89.80
total	158	89.87	102.77	192.66

14 Inventory or material availability and demand provided by NCCR.

5 DESIGN GRADUAL CLOSURE COMPONENTS

In this Chapter, the design of the gradual closure components is presented. This concerns the design of the vertical closure component and the horizontal closure component. The principles presented here and Appendix A5 are the basis for the construction model used to evaluate different scenarios in Chapter 7.

5.1 Main geometry and design parameters

The elevation of the final dam is MSL +8.0 m. This is the elevation after completion of the KALPASAR closure dam, e.g., after applying the final protection at the seaward side against severe wave attack. The closure dam itself can be considered to be completed once the tidal flow has been fully blocked, so prior to the actual finalisation of the dam and for a lower crest height than the final height. The crest width for the closure dam is dependent on the construction method. See Appendix A4, Figure 4.10.

For the vertical closure component, in principle, the crest width of the closure dam can be small in ideal circumstances of low flow intensities and absence of wave action, combined dumping of the closure material from a stationary line (e.g., a cable way). Under the more severe conditions found here, together with the usage of waterborne dumpers, the crest will be more flattened in practice. The over-usage of closure materials remains limited however, as in case of a sharp crest, the envelope of material will encompass lower profiles with a wider crest as well, so the difference will show up in the final highest closure stages. As a best guess a crest width of 20 m has been taken here as realistic. For final design optimisation it is recommended to consider a varying crest width, starting with a wide crest with enclosure bunds and gradually reducing the width in advanced stages.

For horizontal closure component, forward construction with trucks and other dry dumper equipment will determine the required width of the crest (during which the crest level remains at one predetermined level, well above the waterline). From analysis of equipment properties and manoeuvring requirements, a crest width of 20 m has been taken, so coinciding with the crest width of the vertically raised closure component.

The slope angle of the side slopes will theoretically be rather steep as the slopes tend to settle according to the natural slope angle. In practice however, slopes will be less steep, due to dumping inaccuracies and action of flow and waves. To cover these practical aspects a slope angle during closure of 1:3 has been selected for all side slopes, as well as for the front slope of the horizontal closure dam sections.

5.2 Rock stability during construction – current attack

The gradual closure components are designed for current attack using the construction tool (Appendix A4). The tool accounts for increase of maximum current velocities as the construction of the closure progresses.

Flow field characteristics

The head difference over the closure dam will increase with progress of construction. In first stages of closure the head difference is small and flow velocities are governed by the overall flow pattern approaching the closure site, including re-distribution of flow towards areas where closure is lagging behind. This flow situation is not predicted well with the one-dimensional model used here. However, the early stages require only modest stone sizes for stability and are far from critical. This is considered fit for present purpose of determining the feasibility of KALPASAR closure dam. For a detailed design the flow re-distribution needs to be well-predicted making use of a two-dimensional flow model.
As the closure progresses, the hydraulic head will increase, leading to higher flow velocities near the exposed parts of the closure dam initially. Gradually, the flow field becomes more one-dimensional when

closure is done in a balanced way over the various gaps. At this more critical point of construction, the one-dimensional model performance is more accurate.

Beyond this point of construction progress, the conditions for the horizontal closure component and the vertical closure component will divert from each other. The major difference is that for vertical closure component the maximum flow velocity is maximised at progressed stages once flow over the raised crest is becoming super-critical. Such a favourable situation is not occurring for horizontal closure component, for which the maximum flow velocity (near the construction front) will continue to increase. This ultimately leads to excessively large sizes of rock grading to be deployed to achieve full horizontal closure over the present bed of deep channels. Use of excessive sizes of rock grading are not considered for reasons of feasibility. Amongst others, excessive sizes have impact on cost, logistics and construction time.

The above is reflected in the preference for building up the dam vertically at first in the deeper channels and progress with horizontal closure over the strongly raised crest of the dam that has been built up vertically. Early horizontal closure in the shallow areas adjacent to the present coastline, e.g., well outside the deeper channels, is deemed feasible.

Rock stability

For stability of rock, the maximum flow velocity occurring at any given moment of progressed closure for the closure components has been taken as the determining parameter for which stable rock can be selected. Other hydraulic aspects also play a role: curvi-linear flow (particularly important for the horizontal closure), hydraulic roughness and non-equilibrium flow (for hydraulic roughness). Such aspects have been taken into account via simplified stability equations. These stability equations can be considered fit for the present purpose of determining feasibility of KALPASAR closure dam.

Current attack versus wave attack

It is common practice to design the closure components in a feasibility stage for current attack only, for a number of reasons. Most importantly is the increasing complexity of taking into account wave-current interaction. Particularly in advanced stages of closure when strong deformation of waves in increasing flow intensity upon approach to the KALPASAR closure dam when overtopping still occurs. For strong flows, such a deformation will manifest itself by a temporary increase of the water level, adding to the head difference of an advanced closure dam section. Moreover, wave action is more relevant for higher stages of vertical closure than for horizontal closure. The main reason is that the critical section of the vertical closure dam is the crest section and adjoining outer slope, which is most exposed to flow and wave action.

In contrast, the most critical location for a horizontal progressing construction front under flow attack is at a deeper level, where wave action cannot typically penetrate. Also, the consequences for a vertically built-up dam are larger, as the crest level will be (nearly) horizontal over a long section: potential deformation by wave action will subsequently also occur over a vast area. Due to the large tidal range on-site, also the seaward slope of a dam-under-construction can become exposed at low tide and thus can be directly attacked by incoming waves. When the exposed slope is not stable, it will typically transform into a S-shaped profile. For a small variation of water levels a dynamic equilibrium profile may be attained, however here the large tidal range may cause ongoing deformation and loss of material.

For the purpose of determining the feasibility of KALPASAR closure dam, wave attack can be considered a risk factor only, which can well be coped with for the following reasons and with the following measures:

- For the horizontal closure component, wave action is less important. Some deformation will only occur at the dam face that is exposed momentarily, and any deformation is likely to only halt progression up to some hours later during which progress can continue at lower tidal flows and around slack tide. In

practice, the rock size for the horizontal closure component is generally larger than for vertical closure, so there is more redundancy for wave action.

- For the exposed dam crests of vertical closure component, some deformation can occur by wave action at advanced closure stages, leading to a more flattened crest and some rounding-off of the crest in case of overtopping. In case the crest flattens to a lower profile, self-healing can be anticipated, as wave impact reduces strongly with depth. During the vertical build-up process such deformations will normally not be significant, especially not when a wider crest has been taken into account already (e.g., the 20 m of crest width assumed here). In case of wave attack at the outer slopes (at lower tides or at higher elevations of the crest), flattening of the outer slope profile will occur, causing a 'step' of deposited materials near the lower part of the slope. This material can be recovered for the major part if needed and practical.
- For both the horizontal and the vertical closure component, the slope angle assumed is fainter than the natural slopes: i.e., 1:3 [-] compared to about 1:2 [-] or even steeper for natural slopes. This also allows for some deformation and redundancy.

Note that the above applies for active construction fronts. For passive construction fronts, which would be frozen for closure progression for a considerable time (e.g., during monsoon season), risks can be considerable: during monsoon season as the wave attack will intensify (Paragraph 3.3) and the probability of extreme waves increases due to the longer exposure time. Cumulative damage may be considerable for the halted period of construction of say 3 to 4 months. Such risks will be addressed in the feasibility analysis in the Main Report. Additional stabilisation may be required, especially for the vertically built-up dam section. This can be done e.g., by introducing a top layer of more heavy rock at the crest and part of the outer slope prior to monsoon season. Alternatively, a wave-breaking berm can be introduced (see Paragraph 5.4).

It should be noted that for detailed design an integrated approach is recommended, including wave action in all stages of closure and incorporating risks of extreme waves as well. This should also be considered within a wider feasibility and optimisation of construction of the KALPASAR dam as a whole.

On minimum rock gradings

A practical remark should be made here: there is a certain practical lower limit for rock protection materials. For instance: when the size of rock grading determined for advanced stages would be smaller than say a light standard rock grading of 10 - 60 kg, the material should be doubted to be applied at advanced stages for reasons of relatively extreme site conditions, even during construction season, but also because smaller gradings of rock are (i) more prone to screw race and manoeuvring of construction vessels, (ii) the occurrence of seiches that may lead to instantaneous increase of the head difference, (iii) inaccuracies of computational methods for wave and flow prediction and, (iv) placement difficulties in currents due to entrainment.

In such cases, a practical decision is made to select the next higher class than theoretically computed. For 10 - 60 kg this would lead to a choice for 40 - 200 kg, which will be much more robust against the above threats. This applies to the outer slopes of the horizontal closure component, as well as to the crest of the vertical closure component. In the initial stages of vertical closure, the dam body is wide and finer gradings of rock can for instance be used to construct the core of the dam body in between construction bunds of coarser material at the outside. This can be done in the neap tidal cycle, after which the crest should be covered with coarse material before the spring tide cycle sets in, up until the level where the crest and outer slopes are exposed by wave action and the above applies again.

In the following paragraphs a more quantitative elaboration is presented on the potential wave action and consequences in order to better quantify the risks involved. These risks are focused on more extreme

events, e.g., 1/1-year events and more extreme, as these risks are more significant than the 'daily' risks. The daily risks can be considered as more manageable, as follows from the above analysis.

5.3 Rock stability during construction – wave attack

Besides current attack, the closure structure is prone to wave attack during construction, as mentioned before (Paragraph 3.3 and 5.2). Hereafter, the situation of low tide and / or advanced build-up of the vertical closure component is considered in which situation the outer slope will be attacked by waves.

Wave conditions H_s range from 1.40 m (1/1-year return period) to 2.02 m (1/100-year return period). The gradual closure is performed with a maximum size of standard rock grading up to 40 – 200 kg (Paragraph 5.2). Figure 5.1 below indicates the stability of the gradual closure structure for a 1:3 [-] slope. Clearly, the wave conditions exceed the stability of the rock grading designed to withstand current attack during closure construction.

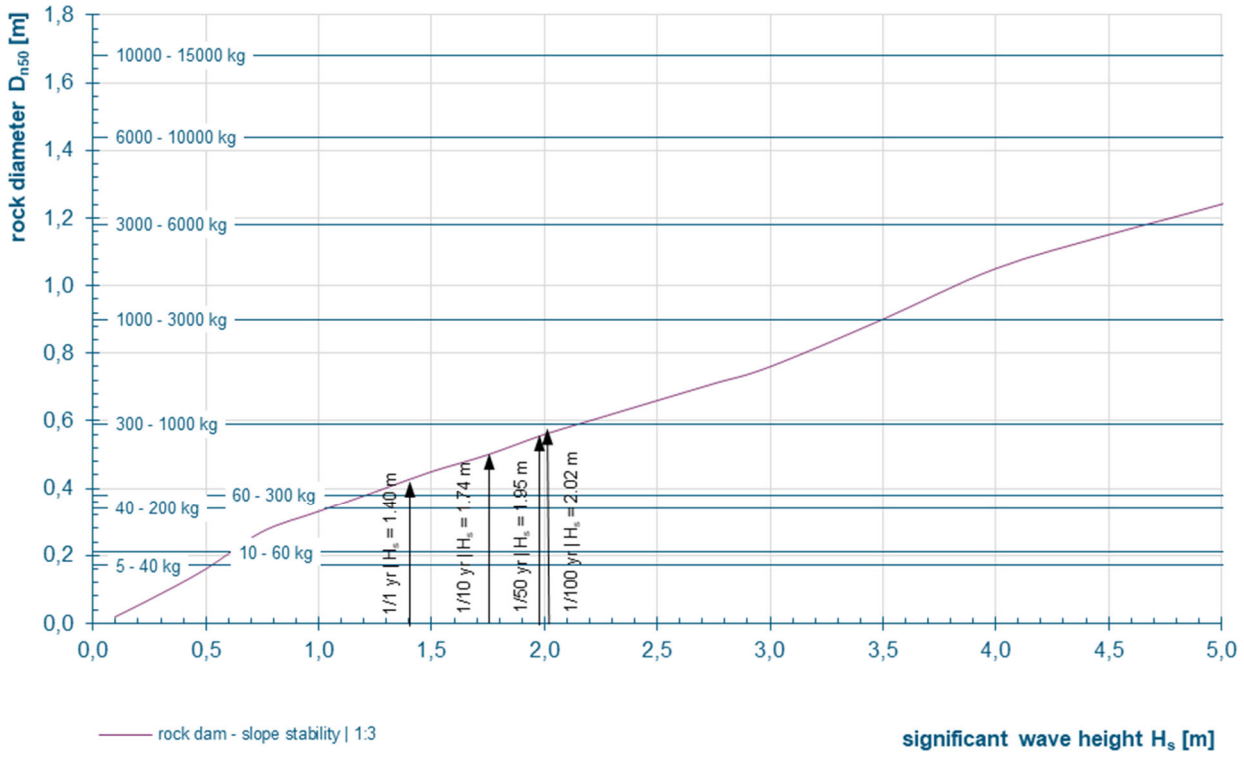


Figure 5.1 Slope stability during construction season

This means that damage / loss of material should be anticipated during the advanced construction stages of the gradual closure until the slopes are finally protected. While this concerns a serious risk, the impact during construction season is deemed to be manageable and limited if the minimum size of rock grading of 40 - 200 kg is adhered to. More extreme conditions that occur during the monsoon season still have to be accounted for (see Paragraph 5.4).

Ideally, parallel construction of the final slope protection for the KALPASAR dam should be pursued. However, this will inevitably draw heavily on construction resources. The various degrees of damage and how to efficiently deal with it during construction is considered qualitatively in the feasibility analysis (see Main Report) but should explicitly be addressed in the detailed design phase. Limited loss of material implies that more volume of rock material is needed to achieve the closure and geometry and that construction will

last longer than calculated. Important is that the closure dam profile remains intact in such a way that no breaching occurs, with significant lowering of the crest.

Limited loss may generally be acceptable since loss of material will be rather localised, with lost rock material typically deposited at a lower level at the outer slope. This material can either be recovered when deemed efficient or left at the bottom of the slope. For a minimum standard rock grading of 40 - 200 kg, we anticipate that the risk of loss of material will be limited as long as no extreme wave events occur (Paragraph 5.4).

5.4 Stability during monsoon season – wave attack

Wave conditions to account for during monsoon are far more severe as is clearly found from Paragraph 3.3. Significant wave heights H_s now range from 1.83 m (1/1-year return period) to 3.97 m (1/100-year return period, see Figure 5.2). The stability of rock material during monsoon season, is therefore more under than outside of the monsoon season. In addition, the 'passive closure front' situation exists by which damage can accumulate because repair is difficult during the monsoon period of four months. Hence, there is more risk that extensive damage occurs or even for the overall integrity of the closure dam is threatened. The key measure here would be to finalise a final protection at the outer slope prior to the onset of the monsoon season for surviving the wave attack during monsoon. However, this again will draw heavily on construction resources and can be highly inefficient.

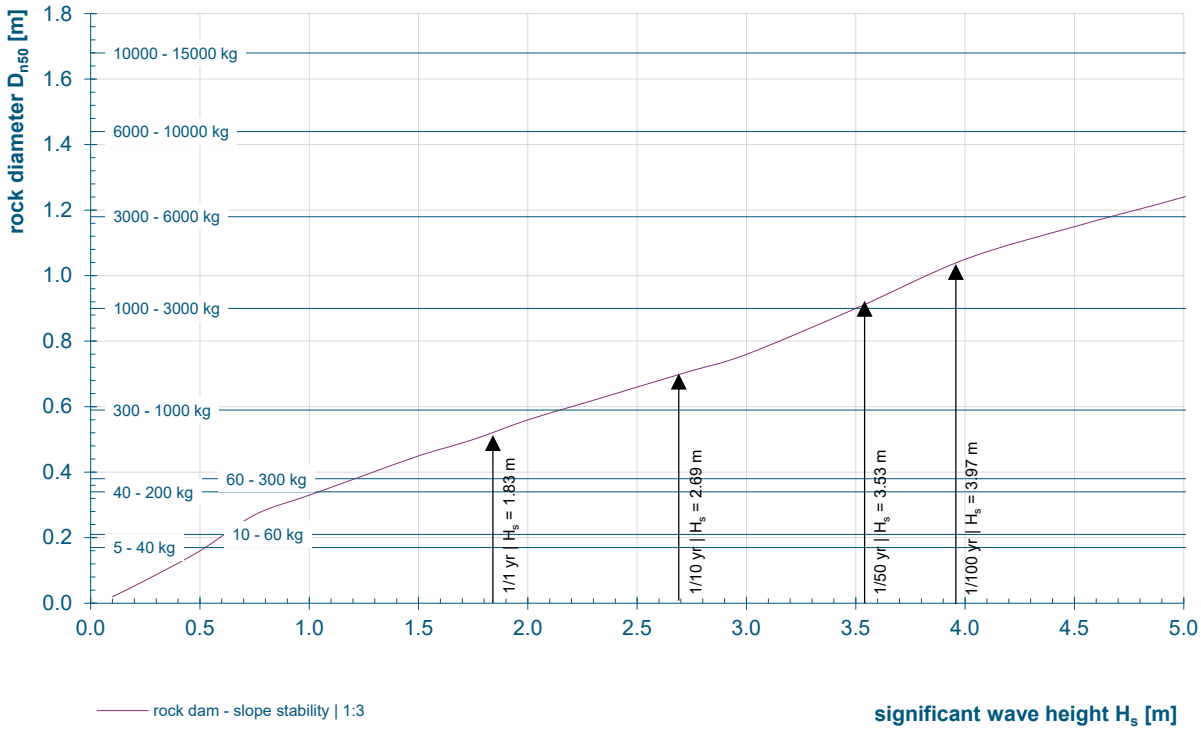


Figure 5.2 Slope stability during monsoon season

In principle, a large length of open concrete closure components is favourable. These open closure elements will be highly wave-resistant, even for extreme waves. This leaves a smaller length of closure dam to be made 'monsoon-resistant'. This will be considered in the feasibility assessment in the Main Report.

As another mitigating measure for the KALPASAR closure dam, to be considered in the detailed design phase, a temporary wave-breaking berm may be applied with very heavy elements (e.g., concrete elements) that are removable, or that can be incorporated in the final design of the seaward slope. To accomodate such a control measure, the closure dam may be provided with a shoulder at a level of say 1 to 2 m below

MSL which allows for easy placement of the wave-breaking berm. An advantage of the berm can be that the construction of the berm may not or hardly interfere with the construction of the closure dam.

5.5 Bed protection

5.5.1 Design principles

A bed protection is required to avoid excessive scouring near the closure works during the closure process. Severe scouring will take place at the downstream side of the closure works (due to the tidal movement scouring will occur on both sides of the closure dam) up from the beginning of closure, but the scouring may increase in intensity for more advanced stages and may reduce for the final stages (for gradual vertical components only). The scouring process is time-dependent and may span a long time (up to tens of years in many cases) to reach equilibrium, dependent on the flow intensity during closure. For KALPASAR closure dam the flow intensity is extreme, so the equilibrium condition will be reached quickly.

The scouring is, obviously, also dependent on the nature of the bed material, i.e., the sensitivity for entrainment, e.g. fine sands will be more susceptible for scouring than coarse gravel beds. For KALPASAR closure dam, the bed generally is fine to very fine and largely cohesionless, so the bed is very susceptible to scouring.

Another issue is the saturation rate of suspended sediment; when this rate is high (as can be expected here), scouring will be relatively less or slower because the pick-up of sediment will be reduced. On the other hand, the flow velocities expected here exceed the validity range for computational methods by far, so this issue will remain uncertain. A fundamental separate study will be required for a more reliable prediction method (in the sense of extension of available knowledge) for scouring in the case of KALPASAR closure dam.

The bed protection length at a certain chainage of the closure dam is the primary design parameter to cope with severe scouring. The scour hole should be so far from the closure dam that geotechnical instability (liquefaction where relevant and/or slope instability) will not endanger the closure works. The flow intensity at the bed protection will be so severe in the case of KALPASAR closure dam that a heavy protection top layer is required to cope with the extreme flow conditions and high turbulence downstream of the reattachment point downstream of the closure works. Hence the bed protection may be a major factor in the closure. Ideally the design of the bed protection should be fully integrated in the total closure dam design, but in practice, the bed protection design often is the closing action in the design of closure works. Here, for the feasibility design, the bed protection is computed and priced for each potential alternative, so insight is gained in the contribution of the required bed protection in the total costs for the closure.

The best scour prediction methods for closure works come from the fundamental knowledge gained from the Eastern Scheldt Storm Surge Barrier in the Netherlands. These methods have been elaborated for the permanent function of the Storm Surge Barrier and hence of the bed protection, as the barrier is open in normal conditions and only closed in case of severe storm surges. For the present feasibility design, these prediction methods have been taken as best knowledge available, even for KALPASAR where the bed protection only has a temporary function, i.e. as long as tidal flow occurs over the location under consideration.

At a follow-up stage of design, the validity of the scour calculations should be checked, and the methods will probably have to be adapted for the extreme conditions (very fine bed material in combination with severe flow conditions and temporary function) at this site.

In summary, scouring is anticipated to be a serious risk to consider. This is also reflected in the design of the bed protection according to the above-mentioned prediction methods. The costs incurred can be excessive (say 1/5 as an order of magnitude of the total costs of closure), whereas the function of the bed protection is only temporary in case of the KALPASAR closure dam. These costs may not be avoidable during the closure process. The present uncertainty is also fuelled due to the substantial differences between the Eastern Scheldt Storm Surge Barrier and the KALPASAR dam, especially in regards to sediment supply and saturation of the flow at the respective sites.

The high costs, as well as the limited knowledge for the KALPASAR (construction) conditions, indicate the urgence for additional research and innovative concepts to reduce the temporary bed protection needed in a further detailed design phase (Paragraph 5.5.2).

5.5.2 Additional research and innovative design opportunities

A major issue for additional research is a further study into the effect of the expected high saturation of the flow for suspended sediment. Whereas for the Eastern Scheldt Storm Surge Barrier the suspended sediment rate of the water that is passing this barrier can be ignored, the rate of suspension of the water that passes the closure works in the case of KALPASAR closure dam may be relatively high. High sediment concentration near the bed is known to supress turbulence and reduce the uptake capacity of the flow. To solve this issue in a practical way the following (three-pronged) approach is suggested:

1. Hindcast predictions with present methods in the Gulf of Khambhat: this can be done from reconnaissance of scour prone structures in the Gulf of Khambhat and measurement of scour depths, structure dimensions, bed and hydraulic characteristics. Types of structures e.g., shipwrecks, spurs, guiding dams, harbour moles (note that large structures with some protection are best suited for use as a reference). The scour depth can then be predicted with existing methods as used here and other methods from literature and compared with the observed scour. The objective here is to have some insight in the ‘scouring effectivity’ of the flow and bed in the Gulf of Khambhat.
2. Laboratory research in a facility that enables high flow velocity sediment transport, in order to assess the amount of sediment transport to be expected for very fine bed material and very high flow velocities, representative for the KALPASAR closure dam construction.
3. Pilot location near the KALPASAR closure dam site. Such a pilot location may be installed at shortest notice (preferably) or may be combined with the earliest placement of a bed protection mattress and be provided with a temporary small length dam. Different prediction methods can be verified, i.e., the present method, as well as adapted methods based on 1 and 2.

Scouring is a response to the intensity of the approach flow over the bed protection. There are various methods to control the scouring and to re-use materials of the bed protection: the scour hole itself can be controlled to some extent, e.g., by controlled dumping of fine stone materials at the upstream slope angle (the stability of which is most endangered and may initiate liquefaction and/or geotechnical instability). This method has been applied for the Eastern Scheldt Storm Surge Barrier and has proven successful. Another method is to control the depth of the scour hole, which however will require a lot of material.
More innovative and promising is to ‘soften’ the transition between fully sand-tight (the bed protection) and sandy bed by enabling reduced sediment transport over the downstream area of the bed protection, by opening up the bed protection for reduced sediment uptake. Some indicative laboratory tests have been performed at Deltares on this solution and results seemed promising, but as far as presently known, no large-scale application has taken place. The idea is that the bed protection will lower itself over the downstream end, and as such reduce the flow attack at the unprotected bed and hence reduce the scouring capacity.

Another promising idea is to break up the bottom flow structure near the edge of the bed protection. By placement of ‘spoilers’ at the bed protection of the Haringvliet Sluices in the Netherlands, surprisingly the scour hole development weakened. The idea is that the bottom eddy (with horizontal axes) that primarily causes the scour hole formation, is being suppressed by breaking up the upstream near-bed flow into smaller scale turbulence. Investigation into the optimum spoiler geometry may be combined with laboratory research as mentioned under 2 in the above.

Re-usage of bed protection materials for other bed protection locations or other gradual closure components. This is highly applicable for shallow areas where initially horizontal closure is being done. Once the front of the horizontal closure has passed, the bed protection is fully sheltered and de facto obsolete, so that material can – if deemed efficient – be recovered and could applied elsewhere. Some reconnaissance on the practicalities involved may give a clue on the feasibility of this idea. The possibility of re-use should be incorporated in the design of the bed protection (for easy handling, strength etcetera, e.g., by applying large steel wire baskets for the rock to be hoisted after usage). For vertical closure locations, some redundancy arises in the advanced stages of closure, i.e., when the scour hole development from earlier stages exceeds the scour hole development capacity of the advanced stage. This may allow for shortening of the bed protection and re-use of the excess bed protection. The re-used material can also be applied for finalisation of the dam body (e.g., protection of slopes at the inner side).

6 OUTLINE DESIGN OF CLOSURE ELEMENTS

An outline design for the closure elements is presented here. The focus of the design lies on fitting the closure elements with an appropriately reliable closure gates rather than the structural design of the closure elements. The most reliable solution is selected and will be applied in both of the two main alternatives identified (Chapter 8).

6.1 General

As focus of the outline design lies on facilitating reliability of the sudden closure first and foremost, the design considers the type of gate and operating mechanisms in detail assuming that any requirements towards the civil components of the design can simply be met. In the design of the civil works, experience with immersed tunnel elements is used to arrive at a robust design with a similar survivability. The immersed tunnel experience is clearly representative as those elements are also prepared off-site, transported over water, and deployed in often severe site conditions, deep down on the seabed. As will be demonstrated from the design this ultimately results in a representative quantity of concrete that allows for price referencing to a level deemed fit for present purpose.

A bridge deck can be added as a modular item should it be necessary to optimise the weight for transportation. As will be demonstrated, the weight of the bridge deck contributes to stability once the element is closed. Likewise, it is at present foreseen for reasons of survivability during construction and ultimately, reliability of the sudden closure that the gates and associated operating mechanism are added once the concrete closure elements have been successfully deployed on their foundation.

Importantly, the construction and supply chain of concrete closure elements is not optimised, and neither is this necessary to determine the feasibility of the KALPASAR closure dam. The construction and supply chain is not optimised at present. When considering construction process within an overall scope for the KALPASAR dam feasibility such can potentially be optimised significantly towards cost and efficiency.

INTERIM CONCLUSIONS – V
The outline design for the concrete closure element presented here is optimised toward reliable performance of the sudden closure. The robust design for the closure performance can be adapted to perform as a spillway element with limited alteration to the concept and with limited additional cost. In doing so it is anticipated that the overall construction of KALPASAR dam will benefit, cost for the sudden closure component will effectively be reduced as part of the cost can be allocated to the spillway and finally, from a hydraulic perspective, performance of the spillway will improve dramatically when positioned in the closure sections, possibly allowing for optimisation of its size relative to its present design. See Paragraph 8.4.

6.2 Outline design of the civil component

The closure component as envisaged is based upon the robust design and experience in the construction of immersed tunnels [15]. Like the concrete closure elements for KALPASAR closure dam, immersed tunnel elements are mass produced off-site and transported to their definitive destination as a floating box before deployment up to depths of 40 m or sometimes 50 m. Elements have a length up to 217 m per box and often accommodate multiple tunnels sections for road and rail transportation. For reference, the largest ones applied in the Fehmarnbelt Fixed Link project weigh up to 73.5 tons. The robust elements have a high survivability. For this reason, a concrete closure element is considered here as well.

The conceptual design shown in Figure 6.1 can be further optimised in the next stage of development. The main dimensions are (b x w x h) 100 m x 30 m x 40 m (height including the bridge deck). Its weight is

15 RHDHV (2023) as TEC lead (Tunnel Engineering Consultants): Busan-Geoje Link Project (3.4 km tunnel), Øresund Link (3.5 km tunnel) and Fehmarnbelt Fixed Link (17.1 km tunnel).

approximately 25 tonnes excluding the vertical lift gates, mechanical and electrical equipment and bridge deck. The weight can be optimised for transportation, but this would entail introducing another step in the deployment process as voids would have to be filled-up for reasons of stability, in which case another dependency on working windows would be introduced which is at present advised against at present. The elements can be mass produced in a dedicated dry dock with a capacity of up to 100 or 200 per year, pending optimisation of the construction process.

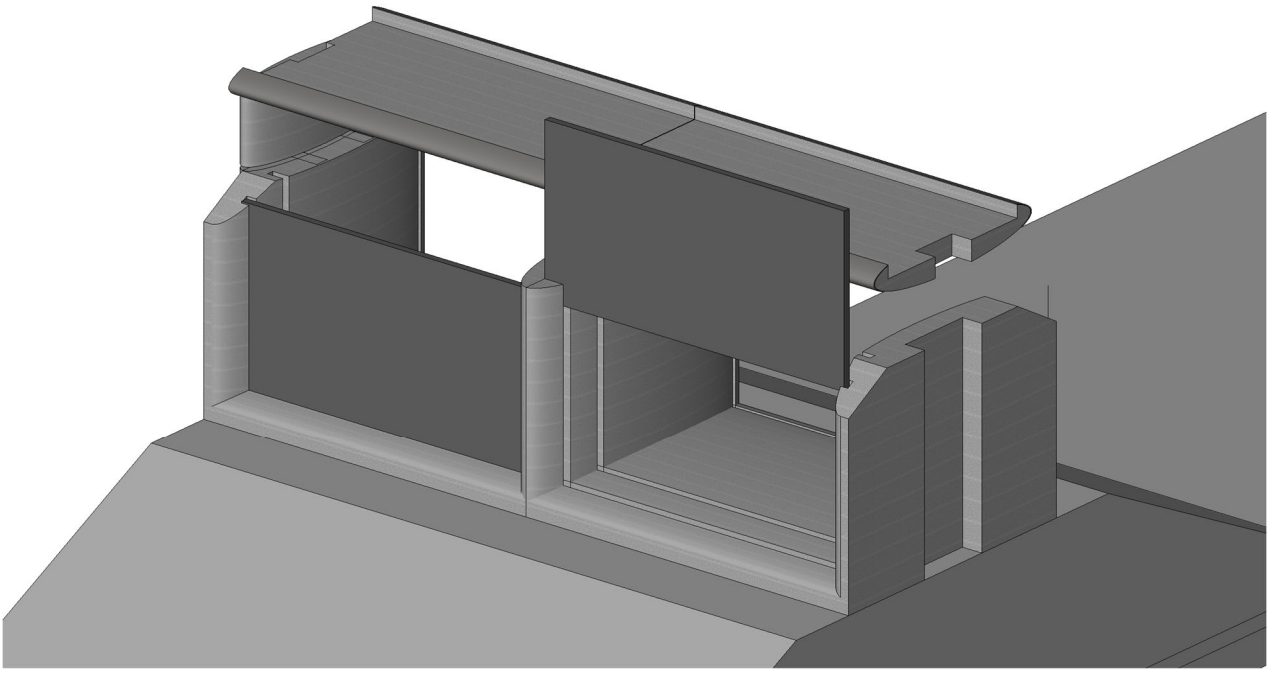


Figure 6.1 3D isometric view of the concrete closure element (two-gate configuration)

Shapes can be hydraulically optimised and interlocking between individual closure elements is possible to ensure alignment of elements and for reasons of stability. The concrete closure element accommodates two vertical lift gates of approximately 50 m wide and a height of 20 m. For reasons of survivability during transport and the time between deployment and actual sudden closure (up to several years), gates are fitted on-site. Likewise, it is at present foreseen that the bridge deck is also fitted post deployment.

The bridge deck can be installed by means of forward construction from landside. Gates are easily installed using readily available offshore lifting cranes such as floating sheerlegs or similar floating equipment. Such equipment has a capacity up to 5,000 tonnes (e.g., Asian Hercules III [16]), which is sufficient to lift and place gates into the slots cast into the side walls. Assistance during lowering can be provided from the bridge deck. If preferred, similar cranes can support or place the bridge deck on the concrete closure element as well as an alternative for forward construction from landside.

For transportation across the Gulf of Khambhat from the construction yard, two main options exist. The first option is to install temporary gates in additional slots cast into the side walls at either side for buoyancy. This will allow the elements to be transported to a parking area and final site of deployment. This option offers less control in terms of positioning and in terms of lowering the closure element on the prepared foundation. The preferred option is to have a purpose-built vessel for lifting the closure elements for transportation and lowering onto its foundation. The level of control for positioning is significantly higher and there is much less dependency on weather conditions. Since at least 100 elements are foreseen and with

16 https://boskalis.com/media/1hvcpx5j/asian_hercules_iii_heavy_lift_vessel.pdf

200 elements a maximum, the efficiency and reliability gained is deemed to outweigh the cost of adapting an existing vessel(s) for this purpose [17].

6.3 Stability of the closure element

The concrete closure element is placed on top of the rock bund and the elements are initially open to allow for the tidal flow to pass through during construction. The open concrete closure element and the rock dam (vertical closure section) should be able to withstand the 1/100-year wave condition, as the during the construction period of the dam it is realistically possible that such a wave condition during the monsoon period can occur (Paragraph 4.4). A deployed concrete closure element is likely to experience several monsoon seasons prior to sudden closure.

The wave loads on the concrete closure element are calculated similar to the approach to hydraulic loading on vertical composite breakwaters. The Goda-Takahashi method is used to determine the wave loads against the vertical wall. In an open condition, the wave loads only act on the exposed faces of the closure element's walls. In a closed condition, the wave load acts along the entire exposed face of gates and walls. Reference is made to Appendix A3 for calculation details and results.

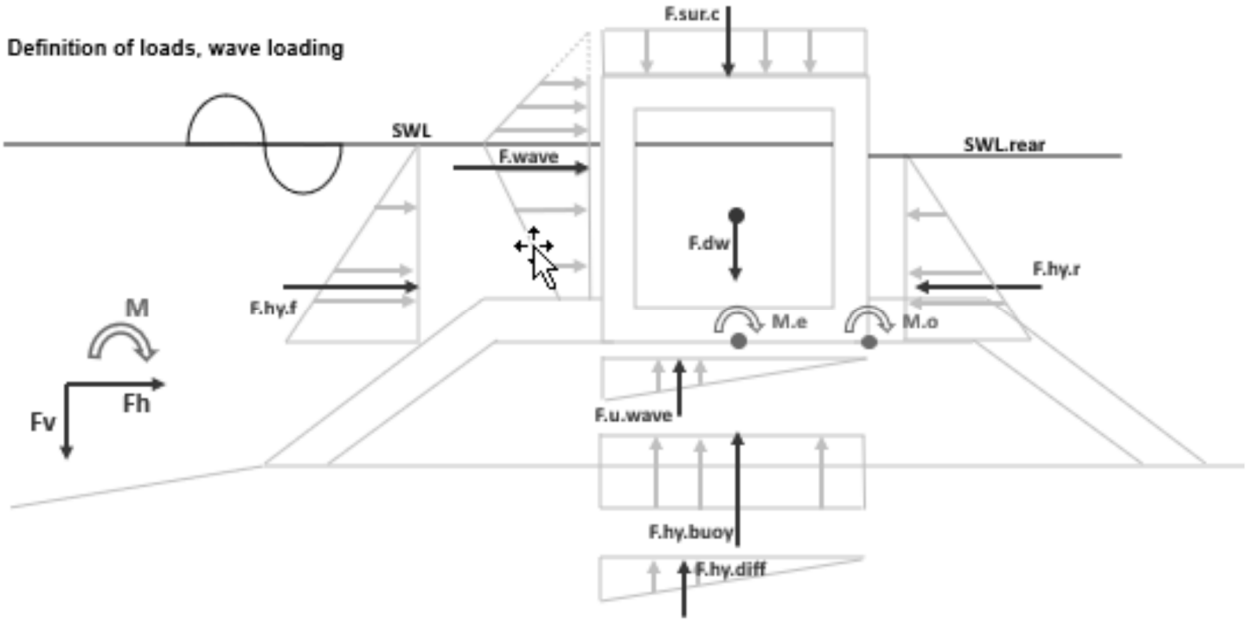


Figure 6.2 Closure element – wave interaction

6.3.1 Stability analysis (closure element)

At low water the assumed dead weight of the structure is more than sufficient to withstand the wave loading. The dead weight of the structure can be reduced to 120,000 kN with the same width of vertical wall (12 m in total for 100 m caisson) in order to keep the unity checks below 1 [-]. The unity check for sliding, $UC_{sliding,}$ is then just below 1 [-].

In case of high water, the assumed dead weight of the structure is more than sufficient to withstand the wave loading. The dead weight of the structure can be reduced to 144,000 kN with the same width of vertical wall (12 m in total for 100 m caisson), to keep the unity checks below 1 [-]. The unity check for sliding,

17 In the case of the Eastern Scheldt Barrier, special purpose vessel Ostrea was built with a lifting capacity of 17,000 tons. Without the gates and bridge deck pre-installed, a similar order of magnitude of weight is to be accounted for, for the concrete closure elements. The need for deployment accuracy and efficiency is similar to what is needed for the KALPASAR closure dam. The number of lifts using the special purpose vessel for deployment of the Eastern Scheldt Barrier's piers was limited to 66 only.

$UC_{sliding,}$ is then just below 1 [-]. However, when the gates are closed the weight of the caisson structure should be heavier to withstand the horizontal wave pressure over the full length of caisson.

Table 6.1 Results of unity checks at low and high water in open condition (1/100-year wave condition)

unity checks wave loading	low water CD -4.0 m [-]	high water CD +6.0 m [-]
unity check gapping, $UC_{gapping}$	0.11	0.13
unity check overturning, $UC_{overturning}$	0.10	0.12
unity check horizontal sliding, $UC_{sliding}$	0.18	0.16

In a closed condition, while there will be added weight from the gates themselves, there will also be added wave loading. The 50 m wide gates are assumed to weigh 500,000 kg, but the dead weight per meter length of the caisson without bridge deck is too low to withstand a 1/100-year wave condition. The horizontal wave force acts along the full length of the concrete closure element and the balancing dead weight force is not sufficient if the caissons are not ballasted with reclamation material or extra weight from a top structure.

The dead weight force of the caisson should be at least 12,000 kN/m to withstand a 1/100-year wave condition of H_s = 4.5 m. This results in a total weight of 122,324 tons. The open closure element with the gates weighs 44,270 tons, so therefore an additional weight of 78,054 tonnes should be added for the closure element to be stable in closed condition under the duress of a 1/100-year wave event.

With the bridge deck added (Figure 6.1), it will add about 29,495 tonnes to the weight of the structure. With a total weight of 44,270 + 29,495 (73,765 tons), the 1/1-year wave condition results in a structure that is just stable with a H_s = 1.82 m during high water level of CD +6.0 m. Which is the governing water level for the stability of the caisson structure. Assuming that in this situation, the water level at the lake side is also CD +6.0 m, the unity checks are as follows:

Table 6.2 Results of unity checks at low and high water in closed condition (1/1-year wave condition)

unity checks wave loading	low water CD -4.0 m [-]	high water CD +6.0 m [-]
Unity check gapping, $UC_{gapping}$	0.10	0.72
Unity check overturning, $UC_{overturning}$	0.13	0.54
Unity check horizontal sliding, $UC_{sliding}$	0.15	0.99

6.3.2 Stability assessment (bed protection)

Apart from structural stability of the concrete closure element with regard to gapping, sliding and overturning, the rock bund underneath is subject to wave loading as well, whereas the closure is optimised towards (increasing) current-attack as closure progress is made (Chapter 5). From Figure 6.3 it is readily found that a 1/100-year event and H_s of 4.5 m cannot be sustained in the closed configuration: standard rock gradings above 1,000 to 3,000 kg would be required, much larger than needed for (practical and efficient) closure. Would a 1/100-year event occur when the closure element is in open configuration, only limited damage should be anticipated as the 40 – 200 kg rock grading used in the gradual closure components is exceeded to a limited extent only.

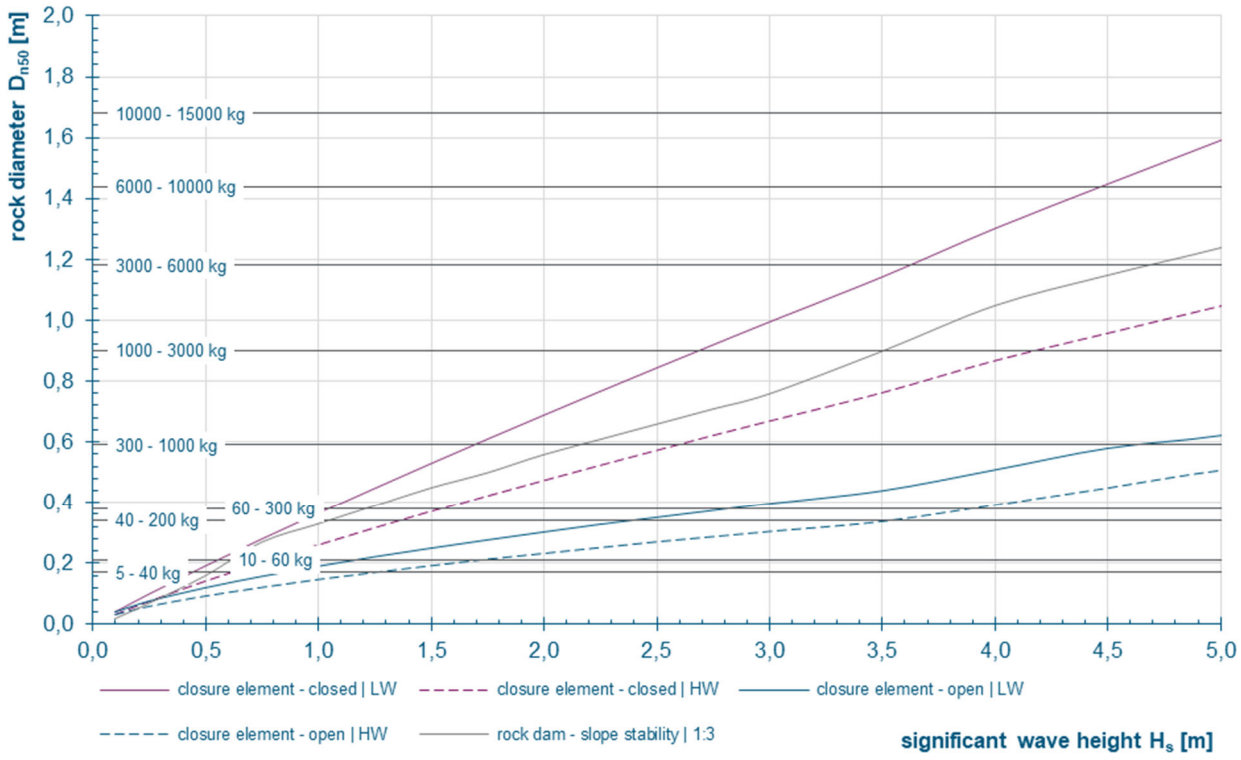


Figure 6.3 Stability of the bed protection adjacent to the open concrete closure element under wave loading

From this, it is clear that once the sudden closure has been performed, the reflected incident wave will increase in height, to an extent that it will exceed a 40 – 200 kg resistance. Thereby confirming again, the need to protect the closed section shortest and ahead of the next monsoon season. Parallel construction should be considered, pending considering of the overall construction process of KALPASAR dam itself (Chapter 5).

6.4 Design of the closure gate

6.4.1 General

Different types of gate have been considered for the closure element that are in principle suitable for the purpose of performing the sudden closure of KALPASAR closure dam. Given the significant length of the final closure gap, a large number of gates have to close simultaneously, with no room for error as the impact of failure on the progressed construction of the KALPASAR closure dam is equally significant and damage to the vertical closure component and bed protection should be expected. Besides damage to the rock structures, failure may lead to closure elements being dislodged from their position and or progressive failure of the sudden closure component. Neither damage is easily repairable and are likely to lead to significant delays and have significant cost impact. Particularly since during a repair effort, hydraulic conditions are worst if a significant section of the final closure gap is also closed (Paragraph 4.3). Therefore, a robust system is needed, despite the fact that the gates and associated closure mechanisms are in principle used only once. In this paragraph, three types of gates are evaluated. It should be noted that this concerns a preliminary evaluation for the purpose of the feasibility study of the KALPASAR closure dam.

6.4.2 General principles

To calculate the probability of failure of the sudden closure for the different gate types (Paragraph 6.4.6), the a gate height of 15 m and 20 m is considered with widths of 20 m, 25 m, 50 m and 100 m, or 5, 4, 2 and 1 gate per concrete closure element of 100 m length respectively.

The bottom level of the gate / concrete closure element is MSL -12.0 m (Figure 6.4). It is further assumed that the gates will be closed at MSL and that the maximum water level (during construction / closure) is MSL +6.0 m. Note that the selection of the water level at the time of closure is subject to further analysis in the next stage, considering the total time needed to perform the sudden closure, travel through the water column of the gate and associated aspects as well as variation in flow velocity (and direction) across the water column. The assumption of MSL for performing the sudden closure is fit for present purpose.

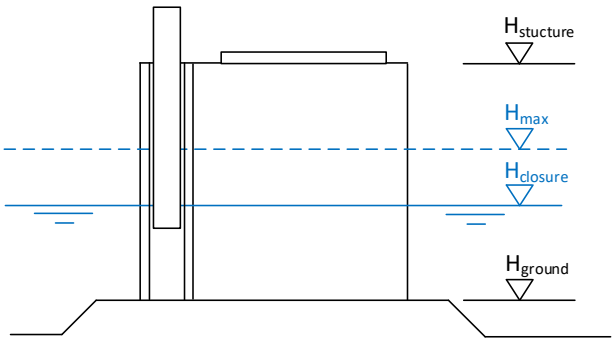


Figure 6.4 Schematised cross-section of the concrete closure element and key elevations (vertical lift gate considered)

The operating mechanism of the gates must be suitable for the sudden closure, implying direct availability of the gates closing. Since the high total number of gates, the reliability of the closure must be exceedingly high. Due to the impact on the closure elements and adjoining structures, a practical fall-back scenario must be possible when one or more gates fail. E.g., a robust performance should be established.

6.4.3 Gate types considered

Three different types of gate have been considered (Figure 6.5): (A) the vertical lift gate, (B) the flap gate and (C) the radial gate. All gates considered meet the basic requirements for robust performance of the sudden closure, pending further evaluation.

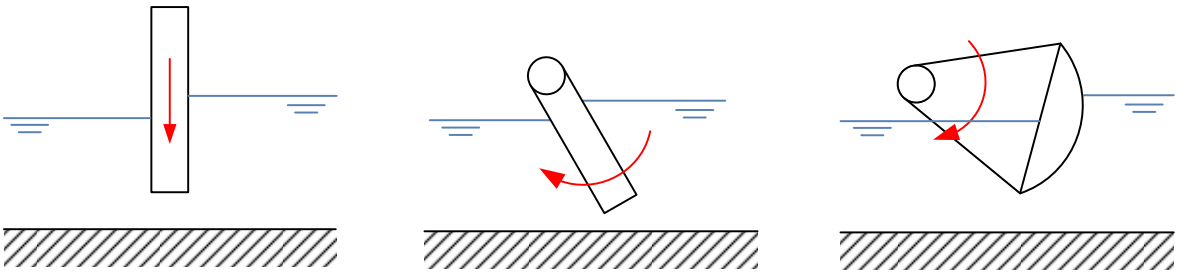


Figure 6.5 Types of gate considered
Left: vertical lift gate, centre: flap gate, right: radial gate

The vertical lift gate is suspended above the wall level and is supported by two concrete columns. This can be easily accommodated in the concrete closure element (Paragraph 6.2). The gate is lowered with a

controlled motion to prevent locking of the gate by two operation mechanisms, each on one side of the gate. The operation mechanism can comprise hydraulic cylinders or a winch system.

A flap gate is a simple steel structure, and its hinge supports are located on top of the walls. Lowering the gate must be controlled with an operation mechanism to prevent damage on the gate and structure. The most suitable operation mechanism for the flap gate is a simple winch system.

The radial gate hinges above water within the concrete closure element. The main advantage of the radial gate is the distribution of hydraulic pressure on the gate, yielding no moments of its pivot. This makes the gate very suitable to close and open under flowing water condition, much more so than the previous two types of gate.

6.4.4 Non-hydraulic design basis

Availability and Reliability: There is a high number of gates which should close simultaneously. When one or more gates fail to close, this can lead to major damage to the dam-structure. Each gate is a part of the system chain. Therefore, an extremely high reliability is required of the gates. Also, the gates should be available when decided to close the dam, which also requires a high availability of the gates. Because the gates will be evaluated for the purpose of feasibility, only unplanned unavailability will be considered.

Maintainability: The gates are necessary for the sudden closure of the dam and will be used only for the sudden closure and the pre-closure tests. During the completion of the dam, the caissons placed with the gates should be maintained for the required high reliability. The gates and the operating mechanism should be easy and safe to maintain. For this reason, it has been decided to post-deployment fitting of the gates once the concrete closure element is deployed at its final position, so avoiding potential damage to the gates and associated component. Gates can be installed as a waterborne activity with support from the bridge deck. The bridge deck is at present also considered to be fitted post-deployment of the concrete closure element. Parts that are temporarily or permanently under water are generally less easily maintainable. The worse the maintainability, the higher the maintenance costs. A design with good maintainability will result in an increase in investment costs (optimisation of Life Cycle Costs).

Sustainability: Standard operating mechanisms are proposed as opposed to mechanisms designed for this specific purpose because the mechanisms are in principle intended for single use. By using standard mechanisms, it is possible to easily remove them and to use them for other purposes. Energy consumption is not considered for the same reason of single use.

Construction time: The total construction time of the control mechanisms can be very important when there is a set timeframe. A short window of roughly one construction season is available for post-deployment fitting of the gates (Paragraph 4.4).

Technical risks: Given the potentially significant impact of failing of the gates, only proven technology is considered. The more experience available with a certain type of lock gate, taking the large dimensions into account, the easier it will be to eliminate risks. This is important, particularly since there is limited experience with sudden closure constructions in general and particularly on the scale of KALPASAR closure dam. Fall-back scenario can be that gates are re-opened in case of a catastrophic failure of one or more gates (internal redundancy) and/or in case it is not possible to close the failing gate by other means (external redundancy). An external measure of redundancy is for instance keeping dredging hoppers on-site during closure that have enough capacity to fill a failed opening in a short time by brute force.

Building costs (investment cost): Often one of the most important criteria are the investment costs. These include the costs of building the gate structure with the operation components and maintenance facilities.

Nevertheless, against the absence of adequate control measures in case of catastrophic failure (Paragraph 6.3), it is for now assumed that the cost of the control mechanisms proposed is justifiable.

Operational and maintenance costs: Operational costs are the costs for energy consumption and control during operation of the control structure. The maintenance costs include the costs for maintaining the gates with their operation mechanism and all other installations required during the operation time of the structure. Such costs are limited to ensuring that the gates fitted post-deployment of the concrete closure element remain intact ahead of actual closure. Gates installed first require a survivability or maintenance effort of one to two years including monsoon seasons. Maintenance cost included pre-closure testing.

Operation mechanism: The operation mechanism is an important element of the gate structure. A simple and robust operation mechanism that can be accommodated with a minimum of requirements and adaptation of the concrete closure element is preferred, e.g., the mechanisms proposed preferably do not require the closure element to be larger than strictly necessary to perform its function as a closure element. Furthermore, a simple mechanism is often more reliable, easy to maintain and repair and often has low investment costs.

6.4.5 Initial evaluation

For the purposes of the feasibility study, it is examined what type of gate is the most suitable for the sudden closure. In the further design phase, a more detailed consideration should be made. To get a pre-considered choice of gate, a shortened and simplified Trade-Off Matrix is presented in Table 6.3. A weighting factor is applied to the different criteria. For the determination of the weighting factors, all criteria are compared to one another. The points-score found gives the ranking of the criteria. The table below shows the assessment and the choice of the weighting factors.

Table 6.3 Weighting factors used in the evaluation

		investment	technical feasibility	operation and maintenance cost	reliability and availability	simplicity of operation mechanism	sustainability	risk / redundancy	sum	weighting factor
1	investment		1	1	0	1	1	0	4	5
2	technical feasibility			1	0	1	1	0	3	4
3	operation and maintenance cost				0	1	1	0	2	3
4	reliability and availability					1	1	1	6	7
5	simplicity of operation mechanism						0	0	0	1
6	sustainability							0	1	2
7	risk / redundancy								5	6

The different types of gate are rated as follows: excellent (3); good (2); adequate (1) and poor / moderate (0). Table 6.4 below presents the result of the assessment of different types of gates and the weighted valuation. The best variant according to the trade-off is the radial gate (C) with a score of 66 out of 84. A variant should obtain a score of at least 70% of the maximum score for it to be suitable solution. With a score of 73 percent, the radial gate (C). The vertical lift gate (A) follows with a score of 70 percent.

Table 6.4 Evaluation of types of gate

				valuation			weighted valuation		
		WF	max score	A	B	C	A	B	C
1	investment	5	15	2	3	1	10	15	5
2	technical feasibility	4	12	2	1	3	8	4	12
3	operation and maintenance cost	3	9	2	3	1	6	9	3
4	reliability and availability	7	21	2	1	3	14	7	21
5	simplicity of operation mechanism	1	3	1	2	3	1	2	3
6	sustainability	4	12	3	2	1	12	8	4
7	risk / redundancy	6	18	2	1	3	12	6	18
	total		**90**	**14**	**13**	**15**	**63**	**51**	**66**
							70%	**57%**	**73%**

Investment: The flap gate scores the best in terms of investment. The flap-gate is very simple in construction with a simple centralised operation mechanism (score 3). The gate of variant A is also a common and simple structure. However, to prevent the wide gate from clamping on both sides of the gates an operation mechanism should be used with an electronic coupling to control the motion of the gates (score 2). The radial gate (C) is a more difficult to build steel structure and the pivot point of the gates will be more difficult in the civil structure, because all the hydraulic pressure will be transferred to these pivots. The operation mechanism can be centralised and simple.

Technical feasibility: The radial gate (C) has the highest score (score 3), followed by the vertical lift gate (score 2). Both types are relative common types of gates used for water reservoirs. The motion of the gates can be controlled adequately during the increasing water level at the open sea. The operation mechanism of the radial gate is independent of the hydraulic pressure on the gate (score 3). The loads on the operation mechanism of the vertical lift gate (A) and the flap gate (B) increase with increasing hydraulic loads. The operation mechanism of the vertical lift gate can withstand higher hydraulic loads than the flap gate. Hence, the latter scores 1 as opposed vertical lift gate which scores 2).

Operation and maintenance costs: The flap gate (B) scores the best on operation and maintenance cost. The operation mechanism is simple and is easy to maintain. The flap gate has a rotation point above water which is relatively easy to inspect. The guidance of the vertical gate (A) is more difficult to inspect and to maintain. Therefore, the flap gate (B) scores 3 and the vertical lift gate scores 2. The operation mechanism of the radial gate (C) is also easy to maintain, however the inspection and perhaps maintenance of the rotation point of the gate is more difficult to inspect and maintain because it is below water level. Therefore, this type of gate scores 1: a rotation point above water level is not possible because then the gate will not fit in the dimensions of the civil structure.

Reliability and availability: The radial gate (C) is rated best with a score of 3, because the operation mechanism is independent of the hydraulic pressure on the gate. The operation mechanism can be carried out relatively simple and a less advanced control system is needed in comparison to the vertical lift gate (A), which is rated 2. The operation mechanism needs an advanced control system on both sides of the gate for synchronous motion of both sides of the gate to prevent locking. But the vertical lift gate (A) scores better than the flap gate (B) because the correct working of the gate can be tested regularly which will be considerably more difficult in case of the flap gate (B). In case of the flap gate, the loads on the operation mechanism are heavily dependent on the hydraulic pressure on the gate, leaving only a short time frame available for testing: testing is therefore already associated with risk of damage during testing alone already.

Simplicity operation mechanism: The flap gate (B) and the radial gate (C) are rated 2: the operation mechanism is very simple and easy to operate and require a less advanced and complex control system as opposed to the vertical lift gate (A). Because of the required accurate synchronous motion of the vertical gate (A), a more complex control system is required with the necessary instrumentation. In case of a failure of the control system it is difficult to proceed with the closing process on manual control of the operation mechanism. Therefore, the vertical gate is rated a score of 1. The radial gate (C) is rated best with a score of 3 because opening the valves for aeration and filling of the floating boxes the filling of the box is very easy. However, the motion will be uncontrolled and there is no correction possible with an operation mechanism when a failure occurs things go wrong.

Sustainability: The radial gate is rated lowest with a score 1. There is no operation mechanism which consumes energy, but the construction has airtight boxes for buoyancy and the civil structure will be wider compared to the others. More material and energy will be required for the construction. The flap gate (B) is rated 2 as while the amount of material is similar to that used in the vertical lift gate (B). The vertical lift gate (A) however allows for use in similar applications while the operation mechanisms used in the flap gate are likely to be downcycled.

Risk / redundancy: The radial gate (C) is rated best with a score of 3. The loads on the operation mechanism are independent of the hydraulic pressure on the gate, so the gate can be tested easily and safely. There is also a natural redundancy available: in case one or more of the gates fail to close, the closed gates can easily be re-opened and return to their initial position. This is also possible with the vertical gate but only to a certain difference in water level. Therefore, the vertical lift gate (A) is rated with a score of 2. The water level difference at which the flap gate can be operated is very low. Hence, there is only a short time-window to close the gate. Re-opening this type of gate again in case of a failing gate will be very difficult. Therefore, the flap gate is rated with a score of 1.

6.4.6 Failure mode analysis (closure element level)

Failure mode analysis is done for the vertical lift gate. There are in principle, to means of operating mechanism that can be considered: (1) the hydraulic cylinder and (2) the wired rope system (Table 6.5). For both systems, a failure mode analysis is done (see Appendix A9) with the results presented in the tables below for gate widths of 25 m and 50 m and so different numbers of gates per concrete closure element and subsequently different numbers of gates in total for different lengths of final closure gap.

Table 6.5 Results of the failure mode analysis for vertical lift gates and hydraulic operating mechanism

	gate configuration		25 m gates			50 m gates		
	length	number of closure elements	number of gates	probability of failure	number of gates failing in closure	number of gates	probability of failure	number of gates failing in closure
1	100 m	1	4	0.0020	1	2	0.0020	1
2	100 m	1	4	0.0018	1	2	0.0018	1

From the failure mode analysis it follows that the hydraulically controlled vertical lift gate is very robust regardless of its width: there is a probability of failure for the hydraulically operated gate of 0.002 [-], while the wired rope operated gate is slightly less reliable with a probability of failure of 0.0018 [-]. The results of the failure mode analysis will be considered to determine the most robust closure alternative in Paragraph 8.4.

7 EVALUATION OF CONSTRUCTION PROCESS

A range of construction scenarios have been analysed using the construction model [3]. The model provides valuable insights to account for when considering closure strategies. The construction model uses the hydrodynamic analysis from Volume 1 as input and thereby accounts for changing hydraulic conditions during construction as discussed in the previous Chapter. The construction model determines the main geometry for the gradual closure components (horizontal and vertical closure component and bed protection (Chapter 5) and associated construction times and cost.

7.1 Construction modelling tool

The construction modelling tool used is a parametric optimisation tool to quickly perform a first-order evaluation of possible closure strategies on costs, construction volumes and construction time. The tool can evaluate and compare different pre-programmed strategies to close a multi-sectional wet cross section using key cost items, e.g., required volume of rock for the closure dam and associated bed protection, as well as different types of construction equipment, e.g., production capacity.

The tool uses the one-dimensional hydraulic model to compute the flow velocities as a function of closure progress. The individual cross sections can be closed by defining their closure method (horizontal, vertical, or sudden), execution phase and construction capacity. Strategic input parameters are defined for all sections, they determine the closure sequence of the system in time. Optimisation is achieved when the strategic input parameters define a closing sequence which minimises the combined cost of all cost factors.

The model further uses three algorithms to quantify the required dam material, bed protection and equipment. These models are described in Appendices A4, A5 and A6 respectively. Based on earlier research, the material model utilises only standard rock gradings for gradual closures and open concrete closure elements (Chapter 6) for the sudden closure. The equipment model (see Appendix A6) uses a range of land-based construction capacities that is representative for different types of construction equipment (e.g., smaller trucks versus larger trucks or even cable-ways and bridge systems) as well as different sizes of waterborne construction equipment (e.g., different sizes of vessel).

Importantly, the model does not explicitly include the cost and construction time of the open concrete closure elements. The time needed for construction and deployment is estimated separately, as is the cost of different lengths of final closure sections, as covered in the previous chapter.

7.2 Summary of salient features per scenario

To understand construction scenario particulars, 12 different scenarios have been evaluated to begin with which are summarised in Table 7.1. These first alternatives serve for general understanding with regard to construction times and cost and the need for optimised logistics. On the basis of these alternatives, viable alternatives have subsequently been selected (Table 7.2) and evaluated to finally identify distinctive alternatives in terms of construction time, construction cost, and robustness of the sudden closure component (Chapter 8).

Table 7.1 Initial set of scenarios (1 – 12)

scenario	total length of horizontal closure [km]	final elevation of vertical closure [m+MSL]	length of final (sudden) closure gap [km]	nr of construction and logistics hubs [-]	description
1	31.3	n/a	0	0	fully horizontal
2	0	10.0	0	0	fully vertical
3	15.0	-10.0	0	0	combined, first closing 15 km horizontal, then closing 17,1 km vertical up to cd + 10 m
4	21.3	n/a	10.0	0	first placing foundation of caissons then closing horizontal while placing the caissons on the foundation. after the gradual closure is completed, perform sudden closure
5	0	-10.0	10.0	0	first placing foundation of caissons, then closing vertically up to cd + 10 m , while placing the caissons on the foundation. after the gradual closure is completed, perform sudden closure
6	8.0	-12.0 (i) -10.0 (ii)	10.0	0	first placing foundation of caissons, the next step is filling the rest of the channels vertically til cd-12 m. thirdly, the horizontal closure of 8 km is performed (4 km on each side). the final step is closing the remaining 13 km vertically. After the gradual closure is completed, perform sudden closure
7	16.3	n/a	15.0	0	first placing foundation of caissons, then closing horizontal while placing the caissons on the foundation. after the gradual closure is completed, perform sudden closure.
8	0	-10.0	15.0	0	first placing foundation of caissons, then closing vertically up to cd + 10 m, while placing the caissons after the gradual closure is completed, perform sudden closure.
9	8.0	-12.0 (i) -10.0 (ii)	15.0	0	first placing foundation of caissons, the next step is filling the rest of the channels vertically til cd-12 m. thirdly, the horizontal closure of 8 km is performed (4 km on each side). the final step is closing the remaining 8 km vertically. after the gradual closure is completed, perform sudden closure
10	29.3	n/a	0	2	first constructing work islands, then closing fully horizontal from 6 fronts (2 land and 4 from the work islands)
11	n/a	-10.0	0	2	first constructing work islands, then closing fully vertical up to cd + 10 m
12	13.0	-10.0	0	2	first constructing work islands, then closing 13 km horizontal (around 2 km per building front, this is not depicted in the figure). the final step is closing vertically over the remaining 16,3 km up to cd + 10 m

Table 7.2 Design set of scenarios 13 - 16

scenario	total length of horizontal closure [km]	final elevation of vertical closure [m+CD]	length of final (sudden) closure gap [km]	nr of construction and logistics hubs [-]	description
13	31.3	-12.0	0	2	vertical gradual closure to MSL -12.0 m, 2 construction and logistic hubs
14	21.3	-12.0	10.0	2	minimum sudden closure gap, 2 construction and logistic hubs
15	16.3	-12.0	15.0	2	
16	11.3	-12.0	20.0	2	maximum sudden closure gap, 2 construction and logistic hubs

7.3 Hydraulic evaluation of scenarios

The scenarios have been evaluated in the one-dimensional model to understand how flow velocities increase as a function of construction progress. The increased velocities are subsequently translated into a rock diameter needed for stability of the closure dam. The results are shown presented in Appendix A8. From the diagrams presented there clearly follows that the maximum progress that can be achieved before rock sizes increase to a size that they are no longer practical to handle, lies around 18 km of horizontal closure.

Beyond that point, rock sizes exceed a standard rock grading of 1 – 3 tonnes (D_{n50} = 0.90 m). Continuing the closure for another 12 km of final closure would have serious implications on logistics in terms of equipment and construction duration at least. Moreover, while according to the client, Indian quarries have agreed to supply rock against a fixed price (e.g., regardless of rock grading), sourcing from foreign quarries (Paragraph 8.3) will likely result in price differentiation per rock grading with the larger ones sometimes substantially more expensive.

7.4 Key assumptions for geometry

Reference is made to Appendix A5. Here, the key assumptions to determine volumes are shown presented. The crest width of 20 m is determined from a practical need to allow for transportation / forward construction to take place efficiently and effectively. From a stability perspective this can be smaller, but logistics and production would suffer. A slope of 1:3 is further assumed, taking into account that whilst a 1:2 slope would suffice in terms of stability, for survivability and robustness during construction (Paragraph 4.3) a conservative 1:3 slope is assumed. The length of bed protection is a function of flow velocity covered in the Appendix.

7.5 Production capacity

Construction duration was determined considering a minimum to maximum range of production capacity for waterborne construction (e.g., the vertical gradual component) and the land-based (forward construction) of the horizontal gradual component. Background to production capacity is given in Appendix A6.2 and A6.3.

In so far as land-based production capacity is concerned, the range has been maximised to use of trucks commonly used in construction activities, as use of rigid-haul or articulated trucks are not considered realistic / practical for use throughout the construction of the horizontal closure component. In case a full gradual closure would be considered, the largest size trucks could be considered but in the case of KALPASAR

closure dam, final closure is done by means of sudden closure. On average and for this feasibility study, the range of horizontal production capacity considered is 4,703 m^3/day to 37,620 m^3/day. Table 7.3 links production capacity per day to fleets of type of equipment.

For waterborne construction, a range of 5,400 m^3/day to 32,400 m^3/s is assumed. Note that in principle, larger vessels can be used, and daily production can be raised accordingly. However, the bulk of the construction is land-based forward construction where more can be gained from increase in capacity.

7.6 Evaluation viable alternatives (scenarios 13 – 16)

Scenarios 13 – 16 are evaluated here in terms of construction duration. The results from the construction model may be used to study and investigate different means of construction equipment and capacity / fleet sizes with the equipment used here as a representative reference. Production capacity versus supply of construction material will be further discussed in Chapter 8. To some extent this also depends on market (construction) capacity and the capacity absorbed in parallel construction of the KALPASAR dam if considered.

From Table 7.3 below it is found that the construction of the vertical closure from the seabed up to a level of MSL -12.0 m takes between 0.8 years to 4.6 years, considering a production capacity of 5,400 m^3/day to 32,400 m^3/day. With an average size of fleets of vessels, the construction should take 1 or 2 construction seasons / years at the most, which is considered reasonable from the point of view of a targeted duration of construction of KALPASAR closure dam up to 7 years.

Table 7.3 Scenario 13 construction years per production capacity

S13	horizontal production [m^3/day]							
vertical production [m^3/day]	4,703	9,405	14,108	18,810	23,513	28,215	32,918	37,620
5,400	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6
10,800	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
16,200	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
21,600	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
27,000	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
32,400	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8

Scenario 14 (Table 7.4) concerns a horizontal closure of 20 km including the construction and logistic support hubs reclaimed, leaving a final closure gap of 10 km. Working at the maximum production capacity (seaborne and land-based), the shortest construction duration is 1.6 construction seasons, notwithstanding market supply capacity for construction material (e.g., rock) and equipment availability. The longest duration is 11.4 construction seasons / years. A reasonable average is 3 construction seasons.

Table 7.4 Scenario 14 construction years per production capacity

S14	horizontal production [m^3/day]							
vertical production [m^3/day]	4,703	9,405	14,108	18,810	23,513	28,215	32,918	37,620
5,400	11.4	7.9	6.8	6.2	5.8	5.6	5.4	5.3
10,800	9.2	5.7	4.5	4.0	3.6	3.4	3.2	3.1
16,200	8.4	5.0	3.8	3.2	2.9	2.6	2.5	2.3
21,600	8.0	4.6	3.4	2.8	2.5	2.3	2.1	2.0
27,000	7.8	4.4	3.2	2.6	2.3	2.0	1.9	1.7

S14	horizontal production [m³/day]							
	4,703	9,405	14,108	18,810	23,513	28,215	32,918	37,620
32,400	7.7	4.2	3.0	2.5	2.1	1.9	1.7	1.6

A 15 km horizontal closure including logistic and construction hubs and 15 km final closure gap remaining. At maximum production capacity the gradual closure components take 1.3 years to construct; at minimum production capacity the duration is 8.9 years. Again, maximum capacity is subject to supply availability. 2 or 3 years is the average number of construction seasons / years of duration.

Table 7.5 Scenario 15 construction years per production capacity

S15	horizontal production [m³/day]							
vertical production [m³/day]	4,703	9,405	14,108	18,810	23,513	28,215	32,918	37,620
5,400	8.9	7.0	6.4	6.1	5.9	5.7	5.6	5.5
10,800	6.3	4.4	3.8	3.5	3.3	3.2	3.1	3.0
16,200	5.5	3.6	2.9	2.6	2.4	2.3	2.2	2.1
21,600	5.0	3.2	2.5	2.2	2.0	1.9	1.8	1.7
27,000	4.8	2.9	2.3	1.9	1.8	1.6	1.5	1.5
32,400	4.6	2.7	2.1	1.8	1.6	1.5	1.4	1.3

In case of a 10 km gradual closure including reclaimed construction and logistic hubs and a final closure gap of 20 km Table 7.6, the duration is obviously the shortest (for the gradual closure components). At maximum capacity, the construction takes between 1.0 and 6.9 years for maximum and minimum production capacity. At average production capacity 2 construction seasons / years would be needed to construct the KALPASAR closure dam.

Table 7.6 Scenario 16 construction years per production capacity

S16	horizontal production [m³/day]							
vertical production [m³/day]	4,703	9,405	14,108	18,810	23,513	28,215	32,918	37,620
5,400	6.9	5.8	5.4	5.2	5.1	5.0	5.0	4.9
10,800	4.5	3.4	3.0	2.9	2.7	2.7	2.6	2.6
16,200	3.7	2.6	2.3	2.1	2.0	1.9	1.8	1.8
21,600	3.3	2.2	1.9	1.7	1.6	1.5	1.4	1.4
27,000	3.1	2.0	1.6	1.5	1.3	1.3	1.2	1.2
32,400	3.0	1.9	1.5	1.3	1.2	1.1	1.1	1.0

The minimum, average and maximum number of years are presented in Table 7.7 for all scenarios for the purpose of illustration and comparison to the selected scenarios 14 - 16. The darker the colour, the longer the duration of construction is. From the result clearly follows that full gradual closure (scenarios 1 - 3 and 10 – 12), not considering the fact that unpractical and large standard rock gradings would be needed to fulfil, requires so much construction time for the KALPASAR closure dam alone, that the overall construction time of 10 – 15 years becomes hardly achievable. This is an important conclusion, that at least with conventional equipment and fleet sizes, this would not be possible, at least not without construction and logistic hubs in play supporting swift construction. This is found from comparison of scenario 1 versus scenario 10 and scenario 3 versus scenario 12: the hubs allow for 6 simultaneous construction fronts and much faster construction for the horizontal closure component.

Table 7.7 Minimum, average and maximum duration of construction (gradual components)
Scenarios 1 – 12 added for illustrative purposes; only scenarios 13 – 16 are used here

scenario	duration of construction (gradual components)			
	min [yrs]	avg [yrs]	max [yrs]	range [yrs]
1	6.7	17.8	49.8	43.1
2	5.9	14.5	35.5	29.6
3	5.8	14.7	38.8	33.0
4	3.5	9.4	26.8	23.3
5	3.9	9.5	23.2	19.3
6	3.8	9.4	23.9	20.2
7	2.4	6.4	18.0	15.5
8	2.7	6.6	16.1	13.4
9	2.6	6.5	16.8	14.2
10	3.5	9.1	24.3	20.8
11	5.7	14.0	34.3	28.6
12	4.5	11.2	28.1	23.6
13	0.8	1.9	4.6	3.8
14	1.6	4.2	11.4	9.8
15	1.3	3.4	8.9	7.6
16	1.0	2.6	6.9	5.9

Note that for selecting distinctive alternatives to assess in terms of feasibility (Chapter 8), the average values are used. The range of construction duration, associated production capacities, price and volume risks are considered in the feasibility assessment in the Main Report.

Table 7.8 below presents the quantities determined per scenario and gradual closure construction component. These serve as input for cost-estimates. Apart from quantities per component, the total annual offtake of rock volume is also presented. The annual offtake is determined by dividing the total volume by the minimum, average and maximum duration of construction as per Table 7.7.

Table 7.8 Minimum, average and maximum duration of construction (gradual components)

	total volume of rock needed					total annual offtake of rock			
	closure dam			bed protection	total	min	avg	max	range
	subtotal	wet	dry	wet					
	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]
1	64.6	0.0	64.6	22.0	86.6	1.7	4.9	12.9	2.0
2	64.6	64.6	0.0	4.8	69.4	2.0	4.8	11.8	2.3
3	64.6	38.8	25.9	6.9	71.5	1.8	4.9	12.4	2.2
4	42.1	4.1	38.0	3.1	45.2	1.7	4.8	12.8	1.9
5	42.1	42.1	0.0	2.1	44.2	1.9	4.7	11.5	2.3
6	42.1	25.3	16.8	2.2	44.3	1.8	4.7	11.7	2.2
7	29.7	8.1	21.6	1.4	31.1	1.7	4.8	12.7	2.0
8	29.7	29.7	0.0	1.1	30.8	1.9	4.7	11.6	2.3

	total volume of rock needed					total annual offtake of rock			
	closure dam			bed protection	total	min	avg	max	range
	subtotal	wet	dry	wet					
	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]
9	29.7	17.8	11.9	1.2	30.9	1.8	4.7	12.0	2.2
10	62.2	0.0	62.2	22.1	84.3	3.5	9.3	23.8	4.1
11	62.2	62.2	0.0	4.8	67.1	2.0	4.8	11.8	2.3
12	62.2	37.3	24.9	5.7	67.9	2.4	6.1	15.1	2.9
13	8.1	8.1	0.0	2.2	9.9	2.2	5.3	13.2	2.6
14	39.1	4.1	35.0	2.2	40.2	3.5	9.6	25.2	4.1
15	27.3	8.1	19.2	1.8	27.3	3.1	8.1	21.0	3.6
16	19.3	8.1	11.2	1.1	19.3	2.8	7.3	18.8	3.3

7.7 Theoretical design versus practical design

Like the hydraulic model, the construction model is one-dimensional. This implies for instance, that the final closure gap always ends up central to the total length of the KALPASAR closure dam, regardless of shallow areas and/or reclaimed construction and logistic hubs. This means that the design achieved by the construction model is theoretical of nature and that while representative results are obtained, the main geometry found must be logically distributed along the length of the closure dam to be representative for what will be built practically. These translations from a theoretical design to a practical design for scenarios 14 to 16 are presented in Volume 4 for the purpose of price / cost referencing. These drawings may also serve as a starting point for detailed evaluation of (intermediate) construction stages and hydraulic evaluation thereof in a two or three-dimensional model in the next stage of development.

8 OPTIMISED ALTERNATIVES

In this Chapter, two alternatives for considering in the overall KALPASAR dam project are presented. One alternative is optimised for the shortest duration of construction. The other is optimised for the lowest cost. Both will be evaluated in more detail in terms of volume, price and time-related risks in the Main Report and both are technically feasible. It will depend on the needs associated with ensuring that the overall KALPASAR dam project is feasible, which of the two is most favourable or at least where emphasis should lie in case of the KALPASAR closure dam. Besides the two alternatives, here the optimised concrete closure element is presented, and failure risk is considered as a function of length of the remaining closure gap for the two alternatives.

8.1 Summary of results and initial comparison

Three optimised alternatives are identified from the simulations to offer a distinct emphasis or choice in terms of construction duration, cost and/or robustness of closure. The three alternatives are considered in a 'resolution of 5 km' of gradual closure and that the maximum gradual closure before rock sizes become unpractical lies around 17 km to 18 km (remaining final closure gap 13 km to 12 km, see Paragraph 7.3). The 5 km resolution is used for the sake of sufficiently distinguishing and identifying risks and impact to address when selecting a preference for construction duration, cost and robustness or hybrid version thereof. The resolution versus practical maximum length of gradual closure will be addressed and nuanced in the Main Report where the final feasibility analysis is presented.

Table 8.1 below presents the three alternatives in terms of main geometry of construction components, construction duration and cost. Further, an initial evaluation is offered in terms of volume risk, price risk and in terms of robustness of final closure to serve as a starting point for the final feasibility analysis in the Main Report. The potential (adverse) impact on the alternative on the KALPASAR dam project as a whole is also presented. Such would for instance be the case if parallel construction is considered and construction of the KALPASAR dam begins after some progress is made with (the gradual component of) the KALPASAR closure dam. Depending on the alternative, this might draw tremendously on construction resources, possibly with higher prices and or longer duration of construction of the KALPASAR dam as a consequence, if not for the KALPASAR closure dam already.

Table 8.1 Summary of results and initial comparison of alternatives

	alternative 1 optimised construction duration	alternative 2 optimised for (direct) cost
scenario	16	14
length of gradual closure [km]	10	20
length of sudden closure [km]	20	10
maximum vertical closure	MSL -12 m	MSL -12 m
volume of rock [mln m^3]	19.3 mln m^3	40.2 mln m^3
direct cost [EUR]	1,392 mln	2,088 mln
risk category		
volume	+	-
price	+	-
time	+	-
KALPASAR dam	+	-

	alternative 1 optimised construction duration	alternative 2 optimised for (direct) cost
robustness of closure	-	+

Again, a more detailed evaluation will be discussed in the Main Report but indicative judgement is already presented here for illustrative purposes: a negative score on robustness for instance does not imply that the risk is unacceptable, rather that the risk needs further and addressing. Likewise, the alternative that is initially the cheapest may, due to the fact that the longer gradual closure component is more prone to loss of material, be more expensive than initially thought. Particularly if it leads to substantial delays which will affect the overall financing scheme for the closure dam and potentially KALPASAR dam as a whole.

8.2 Alternative 1: optimised for the shortest duration of construction

The total construction time is the combined duration of construction of the gradual and sudden closure components. The duration of constructing the sudden closure is determined by the available windows for deployment (Paragraph 4.2). Table 8.2 below provides an estimate of the duration of deployment of 10 km, 15 km and 20 km of concrete closure elements. The maximum duration is obtained if flow velocities need to be below 2.0 m/s; the minimum duration is obtained if flow velocities need to be only below 2.5 m/s for deployment. This means that in case 10 km of sudden closure component and 20 km of (horizontal) gradual component, the time it will take to deploy the concrete closure elements varies between 1.2 and 2.2 years (scenario 14). In case of 20 km of sudden closure component and 10 km of (horizontal) gradual component, the time it will take to deploy the concrete elements varies between 0.9 and 1.0 years.

A single special-purpose lifting vessel is considered for deployment of the concrete closure elements. From the analysis it is concluded that a second vessel will not offer additional benefits in terms of reducing duration of deployment. Even in the case of deploying 20 km of sudden closure elements (scenario 16), the duration is limited to approximately one year, unworkable weather conditions and downtime of the vessel permitting.

Note that in this, it is assumed that deployment begins only once the entire vertical closure has been completed and prepared. This is a reasonable assumption as once the vertical closure is completed; its crest will have to be prepared to serve as a foundation before deploying the concrete closure elements. Pending further study and inclusion of the finalisation of the KALPASAR dam to assess the joint feasibility, optimisation of the construction process may be possible.

Furthermore, the above also assumes that the horizontal gradual closure is done in parallel to the deployment of caissons and so that the number of available windows for deployment shrinks as progress is made. Note that to have the maximum of windows available and to so have the minimum duration of deployment of concrete closure elements, horizontal closure has to be done after all concrete closure elements have been deployed. However, in the interest of a target overall construction time of the KALPASAR closure dam to 7 years approximately, parallel construction is assumed. The number of windows remaining is deemed sufficient pending further optimisation of construction processes of the KALPASAR dam project the next stage of development. The advantage of parallel construction is that the mounting of gates and the bridge deck can be done with land access available.

Table 8.2 Evaluation of scenarios in terms of deployment duration for concrete closure elements

scenario	total length sudden closure	nr of caissons	available windows for deployment		duration of deployment	
			< 2.0 m/s	< 2.5 m/s	< 2.0 m/s	< 2.5 m/s
			min	max	max	min
	[km]	[-]	[-]	[-]	[yrs]	[yrs]
14	10	100	45	85	2.2	1.2
15	15	150	116	185	1.3	0.8
16	20	200	206	234	1.0	0.9

The vertical gradual closure requires 1.9 years to be completed (scenario 12 in Table 7.2 in the previous chapter). Again, it is assumed that it is prepared to serve as a foundation parallel to its construction, pending further study. This means that the duration of construction of the gradual horizontal closure component varies between 0.8 and 2.3 years, for scenario 16 and 14 respectively (Table 8.3). Comparing the duration of full deployment of concrete closure elements with the duration to complete the gradual horizontal component, shows that in case of 10 km and 15 km of sudden closure lengths, the deployment of concrete closure elements is completed before the horizontal gradual closure is finished. In case of a 20 km sudden closure length, the horizontal closure is finished (slightly) ahead of the completion of deployment of the concrete closure elements.

Table 8.3 Evaluation of scenarios in terms of total construction time

scenario	duration to complete		duration to complete deployment of concrete closure elements		order of completion					
	vertical closure to MSL -12 m	horizontal closure	max	min	deployment	horizontal gradual	longest total construction duration	nr of construction seasons	sudden closure in yr from start of vert closure	max construction capacity closure elements needed
	[yrs]	[yrs]	[yrs]	[yrs]	[-]	[-]	[yrs]	[-]	[yr]	[-]
14	1.9	2.3	2.2	1.2	1	2	4.2	5	6	54
15	1.9	1.5	1.3	0.8	1	2	3.4	4	5	81
16	1.9	0.8	1.0	0.9	2	1	2.8	3	4	108

The longest duration of construction is then found from the component that is last finished. In case of scenario 14, the longest duration of construction is 1.9 years for the vertical closure to MSL -12.0 m plus 2.3 years of the horizontal closure or 4.2 years altogether. This means that there are 5 construction seasons to account for in terms of survivability during construction (Paragraph 4.3 and 4.4). If gates and bridge decks are mounted in parallel to deployment of concrete closure elements, it should be possible to complete the fitting thereof within the same number of seasons, safe perhaps another year for trials ahead of sudden closure, pending further analysis.

Theoretically, the sudden closure can be performed in the same year, but because time is needed to finalise and protect the closed concrete closure elements (Paragraph 4.6), the sudden closure has to be done immediately after the next monsoon season so maximum time for this finalisation of the closure dam. Hence,

in case of scenario 14, the sudden closure is done early in the sixth year or construction season. In case of scenario 16, this can be done in the fourth year or construction season, counting from the start of construction of the vertical closure.

This means that the shortest duration of construction is achieved when a case wherein the sudden closure component is maximised. This dictates a maximum production capacity of 108 concrete closure elements per year, assuming that the construction pits are ready at the start of construction of the vertical closure and that production runs parallel to the 1.9 years of vertical closure. For logistical reasons, at least two construction pits are needed (Paragraph 6.2): a production capacity of 108 concrete closure elements is therefore deemed feasible. Pending further optimisation of the sudden closure component in the next stage of development, this a conservative estimate of required production capacity for concrete closure elements.

8.3 Alternative 2: optimised dam configuration for construction cost

To identify a realistic alternative with the lowest construction cost, the combined (direct) cost of the gradual closure components in terms of on-site delivery of the volume of rock needed and total cost of caissons including mechanical and electrical equipment is taken as a representative indicator. The volumes and annual offtake of rock is found from the construction model (Paragraph 7.6). Once identified, the actual overall cost and bill of quantities is prepared in Volume 4.

Importantly, the cost of rock is subject to a price-risk, more so than the cost of concrete closure elements: the total volume of rock is significant and meeting demand will be a challenge for most quarries to maintain construction progress. Since different alternatives require different volumes and have a distinctive duration of construction, the price will ultimately be determined on the basis of annual demand versus supply capacity. Table 8.4 below indicates the price category of rock demand. As an alternative, in the same table an indication of rock material sourced from Oman is presented. Offtake volume depending, Oman prices are between 37.5 percent and 45 percent of prices of rock sourced from India.

Table 8.4 Price per annual offtake of rock for Indian and Oman sourced rock material (on-site delivery and direct cost)

total annual offtake of rock		direct cost of delivery on site	
		Indian sourced	Oman sourced
[mln m^3]		[EUR per m^3]	[EUR per m^3]
0.0	5.0	6.31	2.37
5.0	10.0	6.65	2.59
10.0	15.0	6.87	2.93
15.0	20.0	7.10	3.15
20.0	25.0	7.88	3.60
25.0	30.0	10.14	4.51

It is preferred to source rock material from Indian quarries. While understandable, the sheer volume and transportation from quarries to on-site depots will prove challenging as this will have to be done overland with conventional vehicles. It is further understood from the client that cost will be independent of the actual rock grading required. From experience however, it is understood that for coastal projects, seaborne transportation of Oman sourced rock is cheaper and as well as easier from a logistics point of view. For Oman sourced rock, it is expected that the actual rock grading will have to be factored into the cost. Here it is assumed that for comparative purposes, cost are 25 percent cheaper than Indian sourced rock, pending evaluation in the next stage. In general, it seems that demand can be reasonably met in terms of volume (Paragraph 4.7).

Table 8.5 presents the annual volumes of rock required per alternative in total and per year, based on the duration of construction as determined with the construction model (Paragraph 7.6). With duration of construction for the total gradual components of the KALPASAR closure dam lasting between 4.2 years (scenario 14) to 2.6 years (scenario 16), the corresponding average annual offtake of rock varies between 9.6 mln m^3 and (scenario 14) and 7.3 mln m^3 (scenario 16).

Table 8.5 Required volumes of rock per gradual closure component (total and annual demand)

		total volume of rock needed					total annual offtake of rock			
	average duration of construction (gradual component)	closure dam			bed protection					
		subtotal	wet	dry	wet	total	min	avg	max	range
scenario	[yrs]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]	[mln m^3]
13	1.9	8.1	8.1	0.0	2.2	9.9	2.2	5.3	13.2	2.6
14	4.2	39.1	4.1	35.0	2.2	40.2	3.5	9.6	25.2	4.1
15	3.4	27.3	8.1	19.2	1.8	27.3	3.1	8.1	21.0	3.6
16	2.6	19.3	8.1	11.2	1.1	19.3	2.8	7.3	18.8	3.3

The cost per scenario in terms of on-site delivery of rock is now determined by determining the price per volume of rock (minimum, average and maximum) and multiplying this by the total volume required. This results in the cost range per scenario as stated in Table 8.6 below. In case of an average annual offtake, the cost of rock for the scenarios considered ranges between 128 mln EUR and 267 mln or a factor of 2.1 between the highest and lowest. Importantly, in case a higher production for the gradual closure components would be considered, the difference between the lowest and highest cost increases to a factor of 3.0. Increasing production clearly comes at a cost, obviously more so for the more rock intensive scenarios.

Table 8.6 Overall direct cost of gradual closure components (rock delivered on-site) in EURO

		cost per m^3				total cost gradual components		
		low production			high production	low production long duration		high production short duration
	total	min	avg	max		min	avg	max
	[mln m^3]	[EUR]	[EUR]	[EUR]		[mln EUR]	[mln EUR]	[mln EUR]
13	9.9	6.3	6.6	6.9		62	66	68
14	40.2	6.3	6.6	10.1		253	267	406
15	27.3	6.3	6.6	7.9		172	181	216
16	19.3	6.3	6.6	7.1		122	128	137

Table 8.8 shows the direct cost of gradual components and on-site delivery only versus the duration of construction. A reduction in construction duration of 4.2 years to 1.6 years (factor 0.4 - scenario 14), the cost increases from EUR 267 mln to EUR 406 mln (factor 1.5). For scenario 16, a reduction in construction duration from 2.6 years to 1.0 years (factor 0.4) results in an increase from EUR 128 mln to EUR 137 mln (factor 1.1). This is logical as the volume of rock is far less in scenario 16 than in scenario 14.

Table 8.7 Overall direct cost of gradual closure components (rock delivered on-site) in EURO versus construction time

	total cost gradual components			duration of construction		
	low production long duration		high production short duration			
	min	avg	max	max	avg	min
	[mln EUR]	[mln EUR]	[mln EUR]	[yrs]	[yrs]	[yrs]
13	62	66	68	4.6	1.9	0.8
14	253	267	406	11.4	4.2	1.6
15	172	181	216	8.9	3.4	1.3
16	122	128	137	6.9	2.6	1.0

Table 8.8 presents the minimum, average and maximum direct cost of rock delivered on-site as per above, next to the civil cost and cost of mechanical and electrical equipment, assuming a 100 m concrete closure element equipped with two 50 m steel gates to identify the cheapest scenario. Note that smaller gates can be used which would lead to a less robust solution (Paragraph 6.4). While technically larger gates can be used and would lead to a more robust solution, these are not considered for the purpose of identifying the cheapest alternative as (i) such large gates are hardly ever applied on the scale needed for KALPASAR closure dam and (ii), such large gates would also require mechanical equipment to be manufactured specifically for this project, which is also not considered realistic given that the gates are used only once to perform the sudden closure.

Table 8.8 Overall direct cost of gradual closure components (rock delivered on-site) in EURO

	total cost gradual components			civil	M&E	total cost direct cost		
	low production long duration		high production short duration			low production long duration		high production short duration
	min	avg	max			min	avg	max
	[mln EUR]	[mln EUR]	[mln EUR]	[mln EUR]	[mln EUR]	[mln EUR]	[mln EUR]	[mln EUR]
13	62	66	68					
14	253	267	406	750	375	1,378	1,392	1,531
15	172	181	216	1,124	562	1,858	1,867	1,902
16	122	128	137	1,300	660	2,082	2,088	2,097

Considering the total direct cost for supply of rock on-site, the concrete closure elements and associated mechanical and electrical equipment as a percentage of the civil works cost, scenario 14 (10 km sudden closure with 20 km gradual closure) results in the lowest average cost of EUR 1,392 mln. Scenario 16 leads to the highest average cost (20 km sudden closure with 10 km gradual closure) with EUR 2,088 mln. The difference is EUR 696. In case production would be increased, the difference between the two scenarios is even further reduced to EUR 566 mln.

As mentioned previously, the KALPASAR closure dam project will draw significantly on construction capacity available in the market and particularly in scenario 14, not enough capacity may remain for the (parallel) construction of KALPASAR dam itself. It is anticipated that this will have a similar order of magnitude impact as the impact of increasing production. Considering this, the actual price difference between scenario 14 and 16 will be (much) smaller and closer to the EUR 560 mln mentioned above. While the direct cost found here are only indicative, it once again clearly shows that the feasibility and optimum closure strategy for KALPASAR closure dam is highly dependent on optimisation of the overall construction process of KALPASAR dam in its entirety. Even the alternative with the lowest cost may therefore not necessarily lead to the lowest cost of KALPASAR dam.

Further, for a better comparison, the duration of construction affects the actual cost and for comparison, Net Present Value (NPV) [18] should be considered rather to compare whether a lower cost and longer construction duration outweighs a higher cost and shorter construction duration. On the basis of Table 8.3, the difference in average duration between scenario 14 and 16 is two years only and the cost difference in terms of NPV is unlikely to be significant. However, from the construction model followed that scenario 14 has a range in duration 10 years whereas scenario 16 has a range six years. Depending on the final production capacity adopted, the actual difference in realisation time may well be more than the average two years and there is likely a point where the NPV cost difference becomes less significant than found here. Moreover, the gradual closure components are by their nature more prone to damage / loss of material during construction than the sudden closure component. As a consequence, the scenarios with more length of gradual closure are more at risk of delays and cost-overruns. Several years of delay will lead to much higher cost as well as in delay of generating a revenue stream. Such risks will be further elaborated upon in the feasibility analysis in the DPR / Main Report.

8.4 Alternative 3: optimised concrete closure element for reliability

Optimising for the lowest failure risk of sudden closure depends not only on the length of the final closure gap and so the number of gates per concrete closure element, but also on the type of gate and operation mechanism used. In Paragraph 6.4, a vertical flap gate has been identified as the most suitable option and two different operation mechanisms have been considered, being the hydraulic operation mechanism and a wired rope mechanism. The respective risk of failure determined was 0.0020 [-] and 0.0018 [-] (Table 6.5).

Table 8.9 below presents the probability of failure of the vertical lift gates with a hydraulic operating system as a function of length of final closure gap. In case of a 10 km final closure gap (scenario 14), 100 concrete closure elements would be required. If configured with four gates, the probability of failure is 0.641 [-]. In case of a two-gate configuration, the probability of failure would be 0.321 [-]. In both cases, at the time of performing the sudden closure, it should nevertheless be anticipated that one gate will fail out of the 400 and 200 gates respectively.

Table 8.9 Results of the failure mode analysis for vertical lift gates and hydraulic operating mechanism

	gate configuration			25 m gates			50 m gates	
	length	number of closure elements	number of gates	probability of failure	number of gates failing in closure	number of gates in closure	probability of failure	number of gates failing in closure
	100 m	1	4	0.002	1	2	0.002	1
14	10 km	100	400	0,641	1	200	0,321	1
15	15 km	150	600	0,962	1	300	0,481	1
16	20 km	200	800	1,282	2	400	0,641	1

At a length of 20 km of final closure (scenario 16), the probability of failure increases to 1.282 [-] for a four-gate configuration versus 0.641 [-] in case of a two-gate configuration. In the former case it should be anticipated that two gates fail during sudden closure, whereas in the latter case, only a single gate should be anticipated to fail.

Table 8.10 Results of the failure mode analysis for vertical lift gates and wired rope operating mechanism

18 Note that NPV is fit for present purpose and gives some indication of impact of time and delays, but the method over-simplifies in terms of project finance and bankability. A bankability study for KALPASAR dam as a whole should include these variations in closure strategies determined here.

	gate configuration		25 m gates			50 m gates		
	length	number of closure elements	number of gates	probability of failure	number of gates failing in closure	number of gates in closure	probability of failure	number of gates failing in closure
	100 m	1	4	0.0018	1	2	0.0018	1
14	10 km	100	400	0.7326	1	200	0.3660	1
15	15 km	150	600	1.0989	2	300	0.5492	1
16	20 km	200	800	1.4653	2	400	0.7331	1

In case of wired rope operating mechanism, it is found from Table 8.10 that the probability of failure of gates increases. At a length of 15 km of final closure gap already, two gates should be anticipated to fail where in case of a hydraulic operating mechanism, only one should be anticipated to fail. At 20 km of length of the final closure gap, in case of two gate configuration, the difference in probability of failure is still only marginal and one gate should be anticipated to fail.

It is concluded that hydraulically operated gates of 50 m are the most robust manner of performing the sudden closure. The analysis included redundant control systems, but deliberately not secondary gates. This is fit for the purpose of single use of the gates at the time of sudden closure. Redundancy measures are therefore sought ‘externally’ and practical of nature: the number of gates that fail to close along the 10 km or 20 km is relatively small and manageable.

The nature of KALPASAR project in terms of cost, time and the fact that the closure needs to be performed only once, that at it is deemed justifiable to at the carefully chosen time of sudden closure, have a fleet of dredging hoppers present that quickly reach the gate that has failed and by means of volume, plug the remaining gap with sand or using split-barges stationed nearby with enough supply of rock. After which, time is effectively bought for a more lasting and sustainable stop gap. Pending an integral feasibility assessment of KALPASAR project as whole, even having a fleet of 10 large hoppers or split barges with rock present at the time of closure will be easily justifiable given the stakes and impact of failure. In that respect, again, while the probability of failure increases with the length of the final closure gap, the redundant control measure remains the same and more ‘non-lineair’ and is always justifiable.

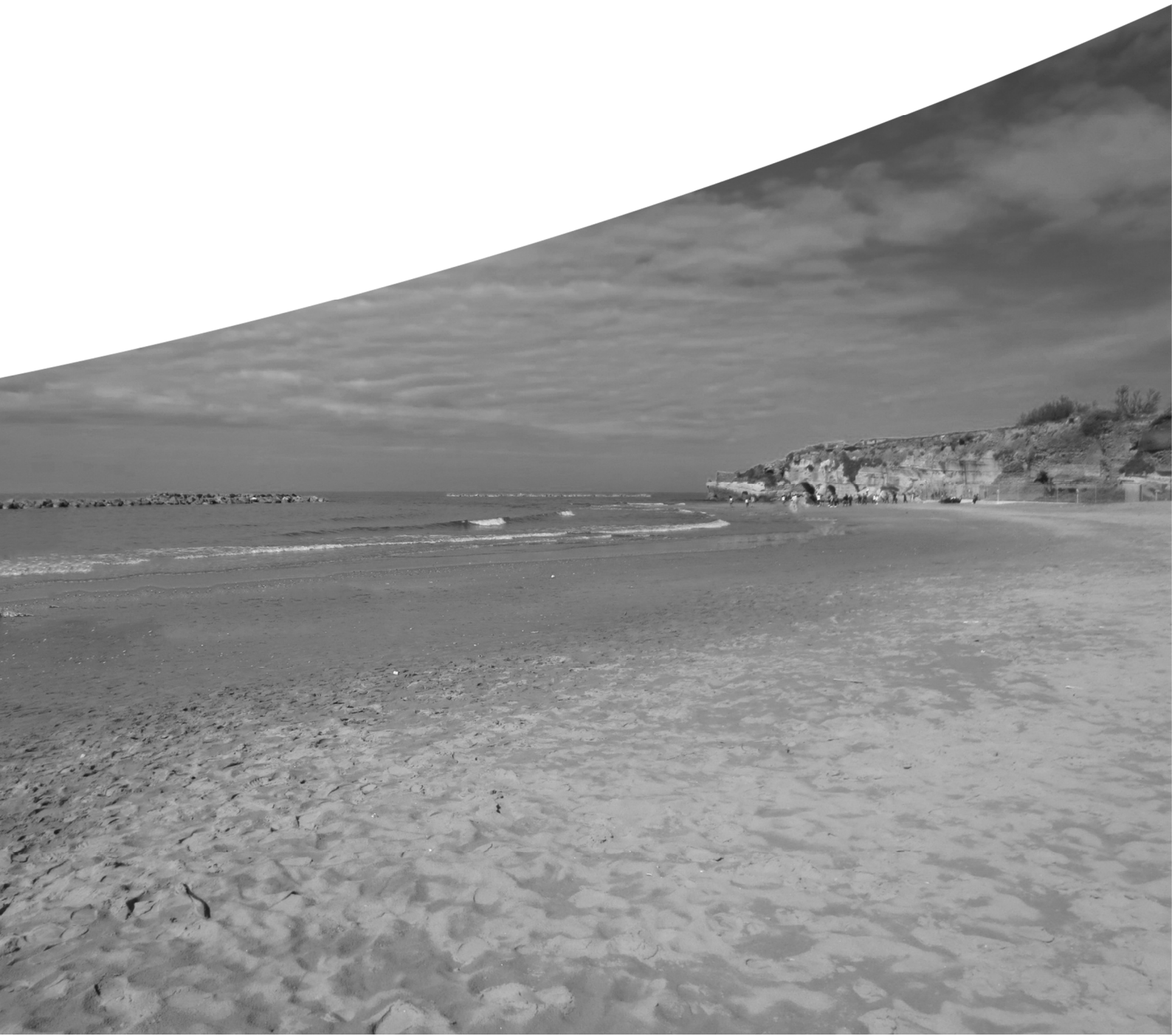
As mentioned in Paragraph 6.1, the concrete closure element can be adapted to perform as a spillway component with relatively little effort and against relatively little extra cost in terms of closure elements, although the bed protection would have to be upgraded in its immediate vicinity. At present, it is understood that the spillway has an envisaged length of 2.2 km, equivalent to 22 100 m concrete closure elements. The spillway would as such comprise of 40 vertical lift gates. In terms of reliability of sudden closure, combining the final closure section with the spillway would result in a higher reliability. Moreover, when incorporated in the final closure section, the spillway can be used to reduce hydraulic loads on a failed section, so adding additional redundancy. The latter much more so as flow is concentrated in the deeper channels as opposed to the shallow area where the spillway is foreseen at present.

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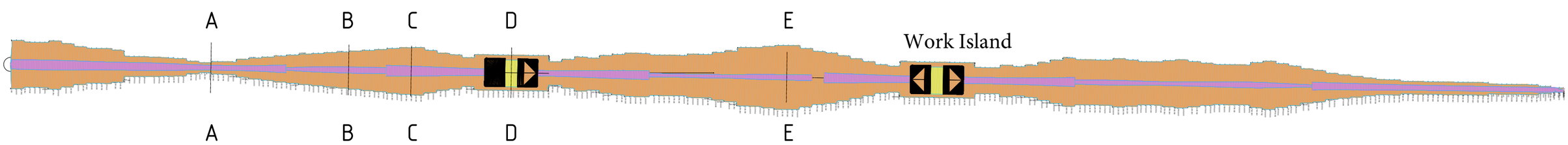
APPENDICES

Volume 2: Technical Report

Kalpasar Closure Dam Feasibility Design



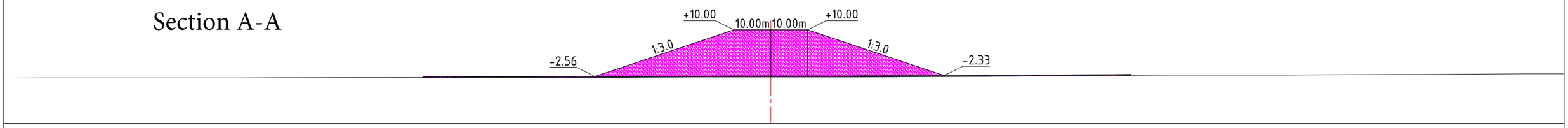
A1 CLOSURE DRAWING



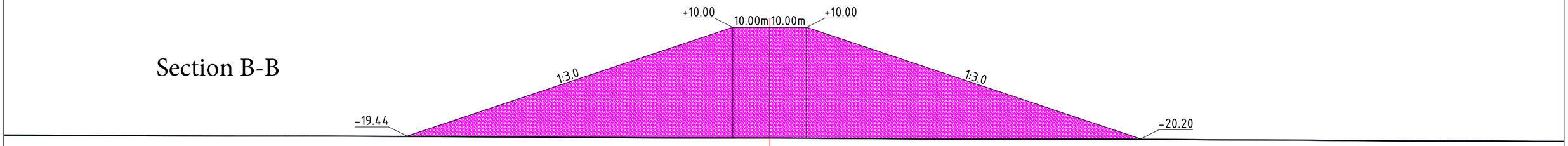
Closure Dam Plan View

Closure Dam Cross Sections

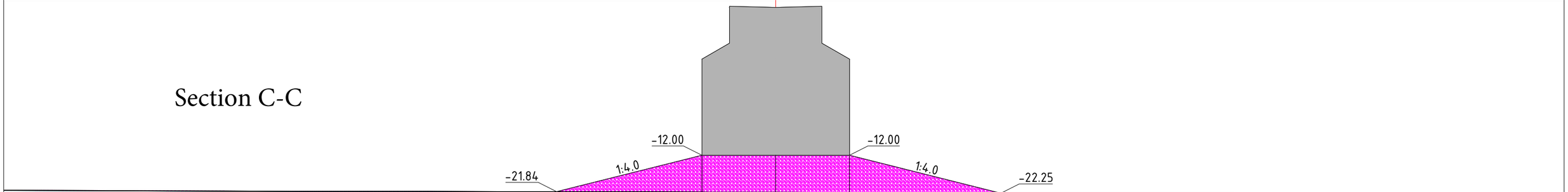
Section A-A



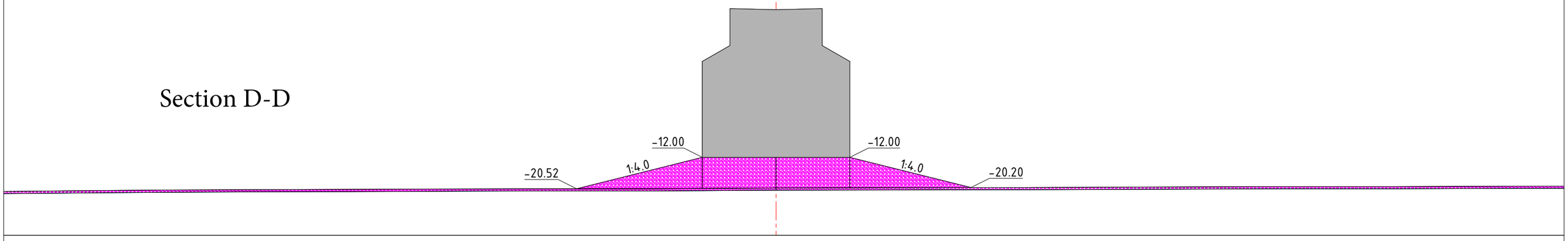
Section B-B



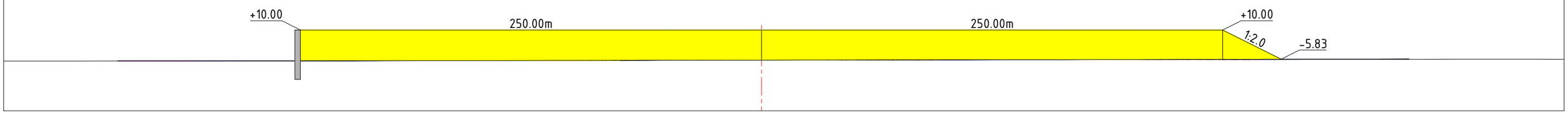
Section C-C



Section D-D



Section E-E



A2 WAVE CLIMATE ANALYSIS

A2.1 NOAA data at point C

The NOAA database consists at this location consists of 31 years of data, from which a peak-over-threshold method is taken to do the extreme value analysis (EVA). See the data and the selected events in the figure below.

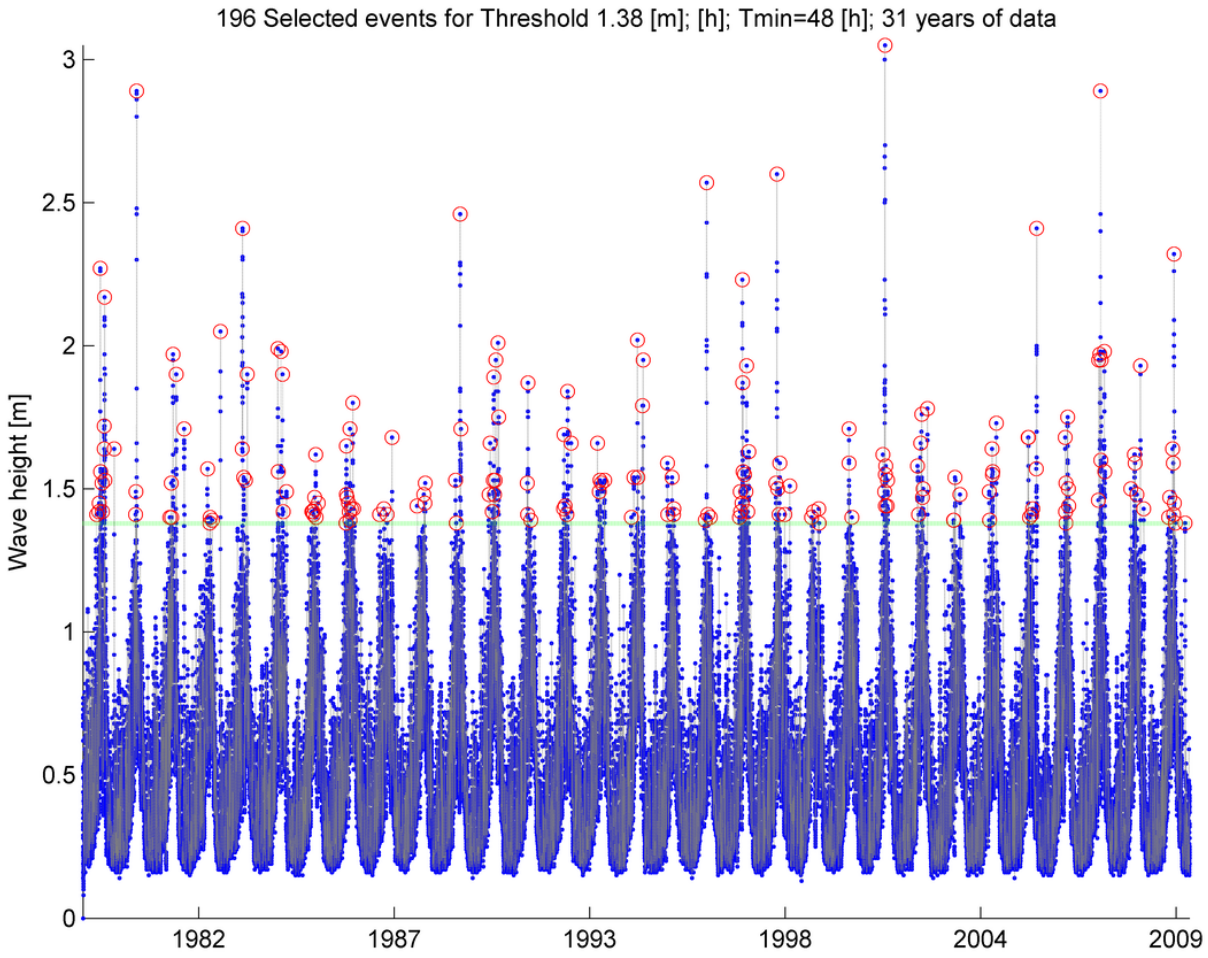


Figure 2.1 Peak over threshold values for 31 years of data at NOAA point C

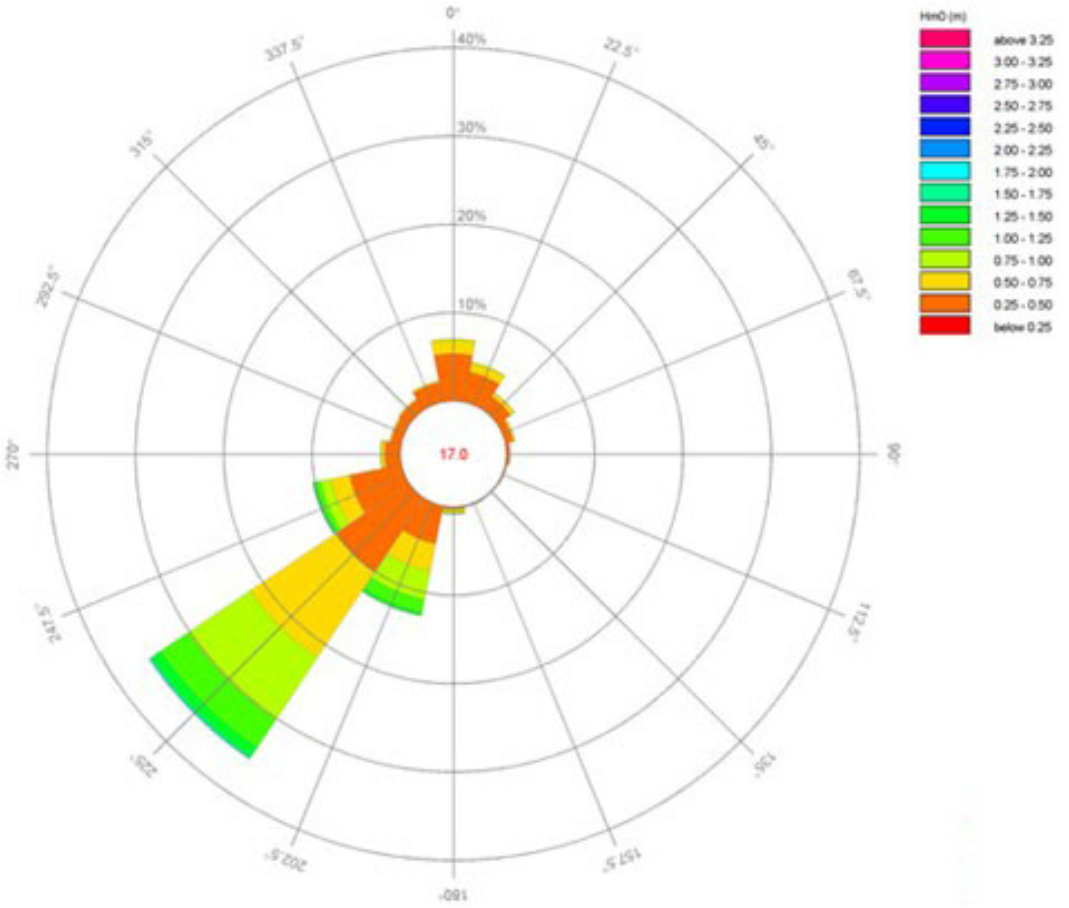


Figure 2.2 Wave rose for H_s at NOAA point C year-round

The highest waves are from the SW during the Monsoon period. In the following figure the wave roses are shown during all months. From this plot it can be seen that during the months from October to March it is relatively calm, and in the months from April until September higher waves are to be expected, mainly from the Southwest.

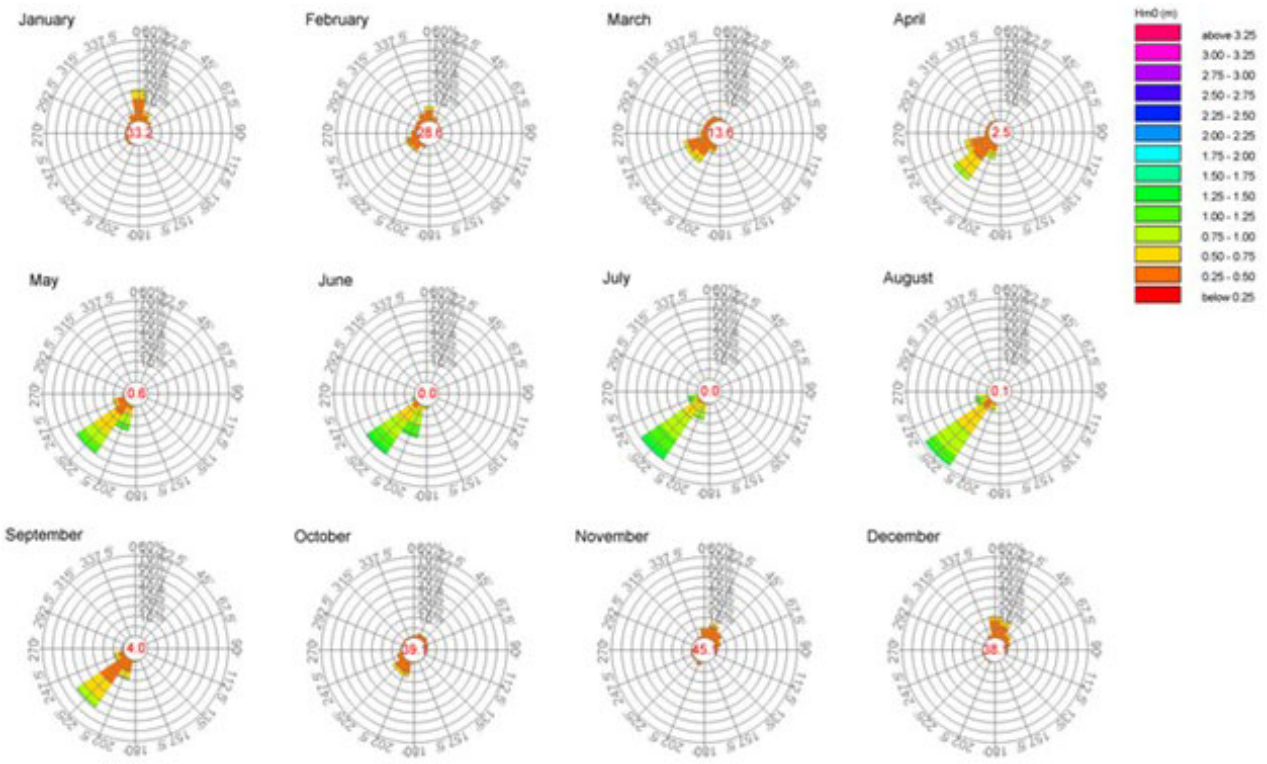


Figure 2.3 Wave rose for H_s at NOAA point C per month

A2.2 EVA results at point C

With the peak-over-threshold method and the selected events, a General Pareto Distribution is fitted, see the figure below:

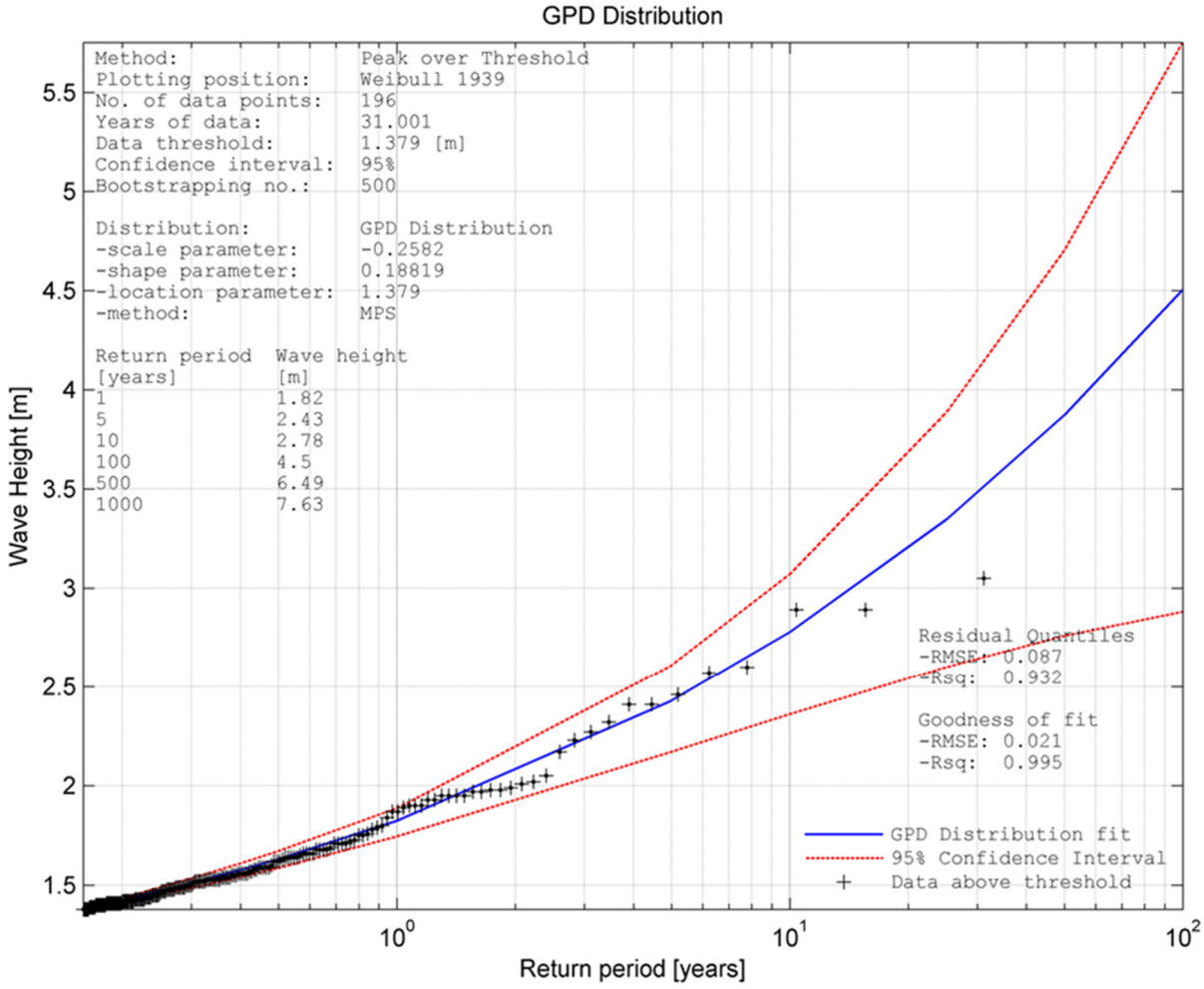


Figure 2.4 EVA year-round at point C

Table 2.11 EVA at NOAA data point C (year-round)

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/0.2	1.43	5.38	1.41	1.42	1.42	1.43	1.44	1.44	1.43
1/0.3	1.51	5.54	1.48	1.49	1.49	1.51	1.54	1.54	1.53
1/0.4	1.58	5.66	1.53	1.54	1.55	1.58	1.62	1.61	1.61
1/0.5	1.63	5.76	1.57	1.59	1.59	1.63	1.69	1.67	1.67
1/1	1.82	6.09	1.72	1.75	1.76	1.82	1.91	1.89	1.88
1/5	2.43	7.03	2.10	2.17	2.21	2.39	2.68	2.61	2.57
1/10	2.78	7.51	2.25	2.36	2.42	2.72	3.18	3.07	3.01
1/25	3.34	8.24	2.40	2.60	2.70	3.24	4.09	3.89	3.78
1/50	3.87	8.87	2.45	2.76	2.92	3.73	5.01	4.70	4.55

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/100	4.50	9.57	2.43	2.88	3.11	4.32	6.21	5.75	5.52
1/250	5.53	10.60	2.21	2.94	3.32	5.27	8.34	7.60	7.22
1/500	6.49	11.48	1.83	2.87	3.41	6.17	10.51	9.47	8.93
1/1000	7.63	12.45	1.18	2.64	3.39	7.25	13.33	11.87	11.11

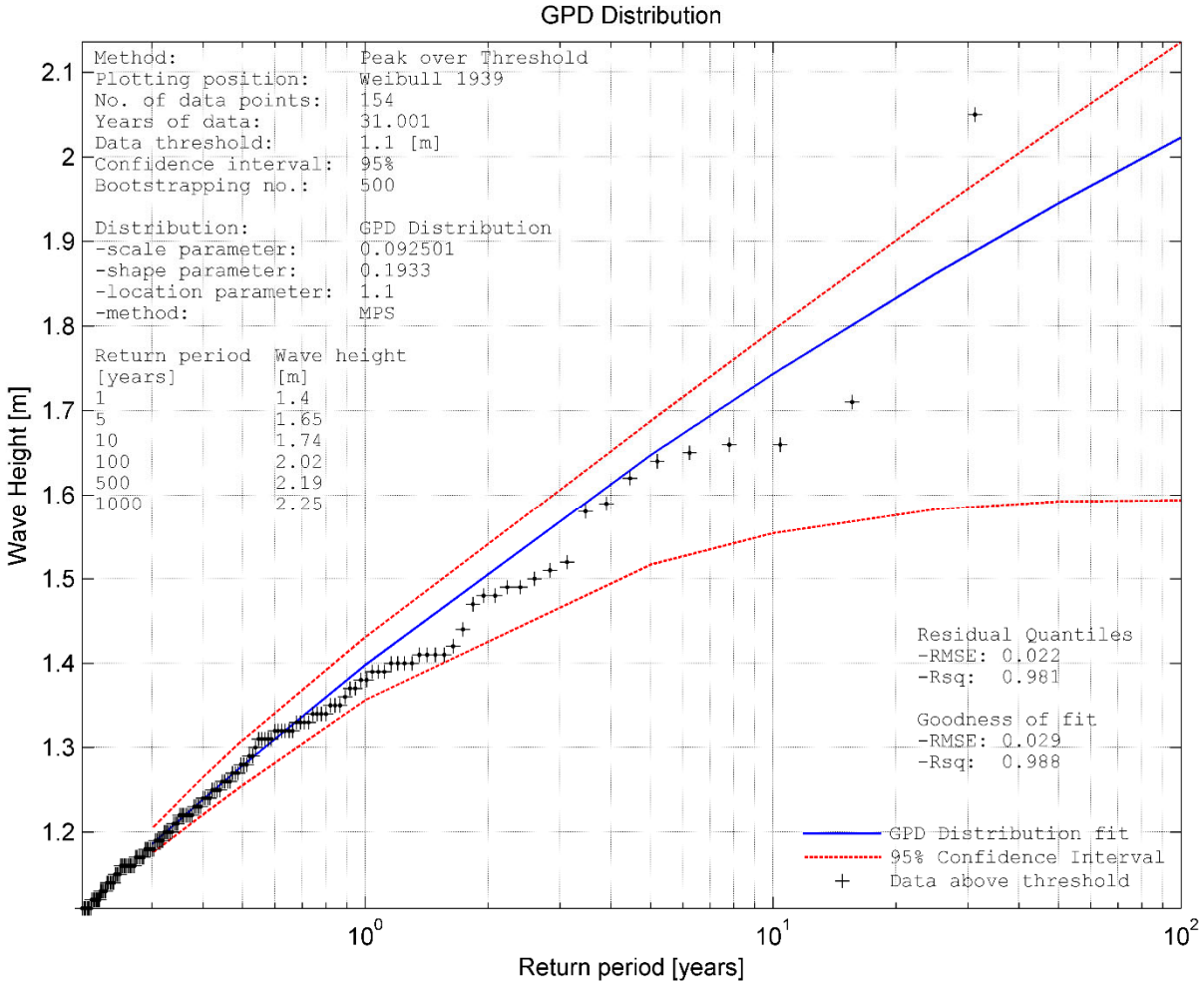


Figure 2.5 EVA outside monsoon season at point C

Table 2.12 EVA at NOAA data point C (outside monsoon season)

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/0.2									
1/0.3	1.19	4.97	1.17	1.17	1.18	1.19	1.21	1.20	1.20

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/0.4	1.24	5.08	1.21	1.22	1.22	1.24	1.27	1.27	1.26
1/0.5	1.28	5.16	1.25	1.26	1.26	1.28	1.32	1.31	1.31
1/1	1.40	5.39	1.34	1.36	1.36	1.40	1.45	1.43	1.43
1/5	1.65	5.86	1.49	1.52	1.53	1.61	1.72	1.69	1.68
1/10	1.74	6.02	1.52	1.56	1.58	1.68	1.84	1.80	1.78
1/25	1.86	6.23	1.53	1.59	1.61	1.77	2.00	1.95	1.92
1/50	1.95	6.36	1.52	1.59	1.63	1.82	2.12	2.05	2.01
1/100	2.02	6.49	1.51	1.59	1.64	1.87	2.24	2.15	2.11
1/250	2.12	6.64	1.47	1.59	1.64	1.93	2.39	2.28	2.23
1/500	2.19	6.74	1.44	1.57	1.64	1.98	2.51	2.38	2.31
1/1000	2.25	6.84	1.41	1.55	1.63	2.01	2.62	2.47	2.40

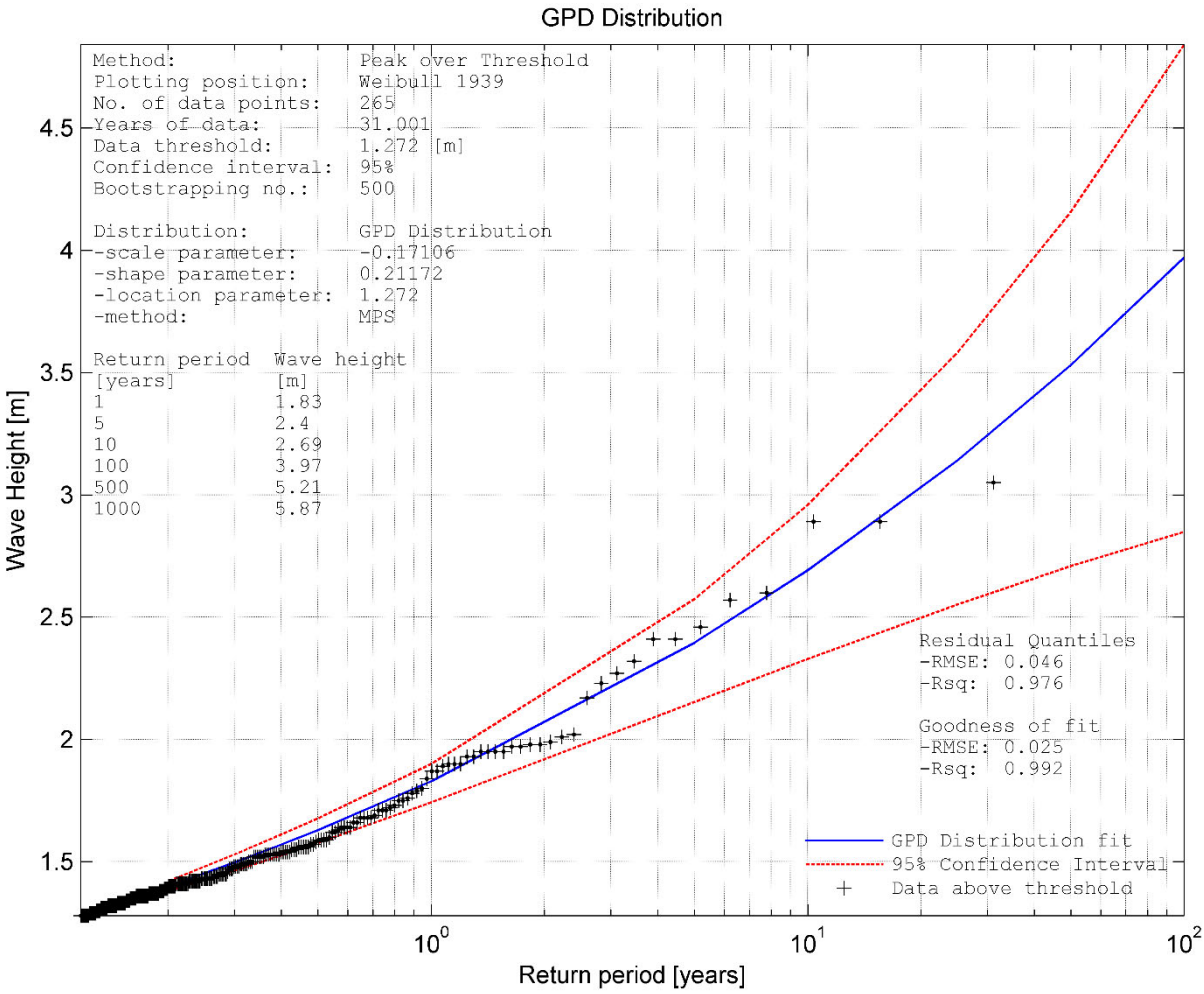


Figure 2.6 EVA during monsoon season at point C

Table 2.13 EVA at NOAA data point C (during monsoon season)

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/0.2	1.40	5.33	1.37	1.38	1.38	1.40	1.43	1.42	1.42
1/0.3	1.50	5.51	1.45	1.46	1.47	1.50	1.54	1.53	1.52
1/0.4	1.57	5.65	1.51	1.53	1.53	1.57	1.62	1.61	1.60
1/0.5	1.63	5.75	1.56	1.58	1.58	1.63	1.69	1.68	1.67
1/1	1.83	6.10	1.72	1.74	1.76	1.82	1.92	1.90	1.89
1/5	2.40	6.98	2.09	2.15	2.19	2.36	2.64	2.57	2.54
1/10	2.69	7.40	2.23	2.33	2.38	2.64	3.06	2.96	2.91
1/25	3.14	7.99	2.39	2.55	2.64	3.07	3.74	3.58	3.50
1/50	3.53	8.47	2.48	2.71	2.83	3.43	4.39	4.16	4.04

Q_s	H_s [m]	T_p* [s]	99%	95%	90%	50%	-90%	-95%	-99%
1/100	3.97	8.98	2.53	2.85	3.01	3.85	5.16	4.84	4.68
1/250	4.64	9.71	2.53	3.00	3.24	4.47	6.41	5.95	5.71
1/500	5.21	10.29	2.46	3.07	3.39	5.02	7.58	6.96	6.65
1/1000	5.87	10.92	2.30	3.10	3.51	5.64	8.97	8.17	7.76

A3 STABILITY ANALYSIS CONCRETE CLOSURE ELEMENT

A3.1 Wave loads and hydrostatic loads

The wave loads on the concrete closure element are determined using the Mathcad sheet “Hydraulic loading vertical composite breakwater”. The Goda-Takahashi method is used to determine the wave loads against the vertical wall. This load will only occur to the walls of the open caisson, and to the whole frontside of the closed caisson.

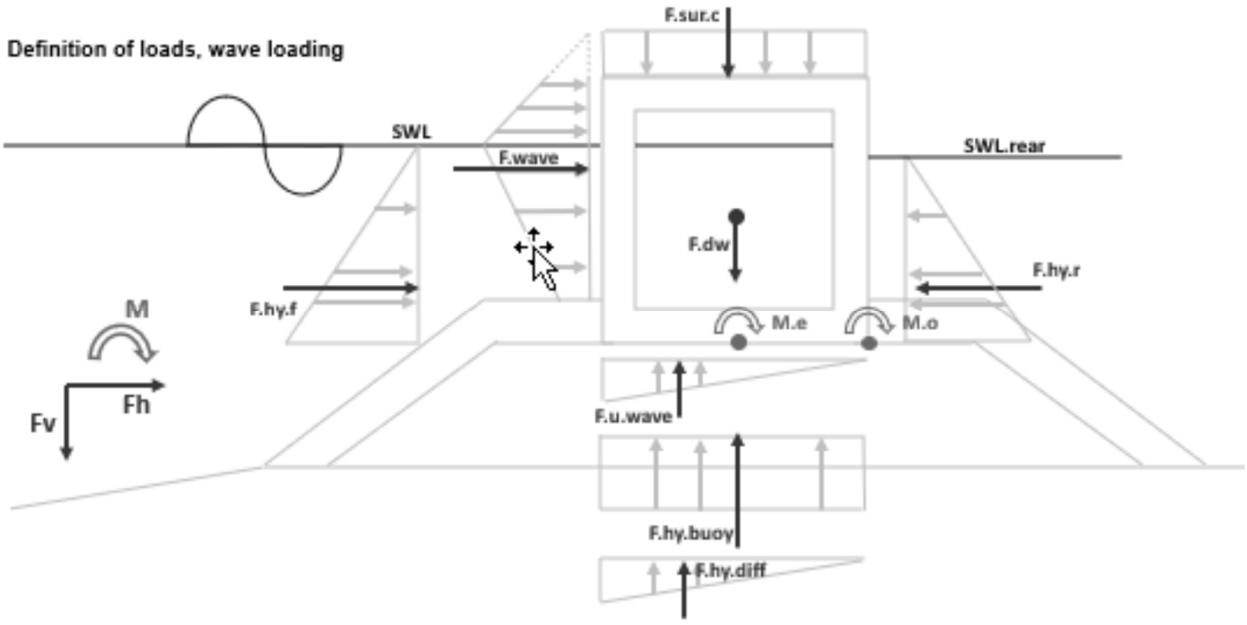


Figure 3.7 Wave – structure interaction

The results are as follows (open and closed caisson with no differential in water level between front and rear):

Table 3.14 Wave forces from wave – structure interaction

Wave loads	Low water -4m CD	High water +6m CD
Wave force, F_{wave}	1729 kN/m	1400 kN/m
Wave moment, M_{wave}	13243 kNm/m	14322 kNm/m
Wave uplift force, $F_{wave.u}$	-805 kN/m	-950 kN/m
Wave uplift moment, $M_{wave.u.e}$	4027 kNm/m	4749 kNm/m
Wave uplift moment, $M_{wave.u.o}$	16109 kNm/m	18997 kNm/m

Table 3.15 Hydrostatic forces from wave – structure interaction (open)

Hydrostatic loads horizontal	Low water -4m CD	High water +6m CD
Hydrostatic force, $F_{hy.f}$	323 kN/m	1636 kN/m
Hydrostatic force, $F_{hy.r}$	-323 kN/m	-1636 kN/m
Hydrostatic moment, $M_{hy.f}$	862 kNm/m	9818 kNm/m
Hydrostatic moment, $M_{hy.r}$	-862 kNm/m	-9818 kNm/m
Uplift buoyancy (permanent), $F_{hy.buoy}$	-2424 kN/m	-5455 kN/m

Hydrostatic loads horizontal	Low water -4m CD	High water +6m CD
Uplift differential hydrostatic (variable), $F_{hy.diff}$	0 kN/m	0 kN/m
Uplift buoyancy moment, $M_{hy.buoy.e}$	0 kNm/m	0 kNm/m
Uplift buoyancy moment, $M_{hy.buoy.o}$	36364 kNm/m	81820 kNm/m
Uplift differential moment, $M_{hy.diff.e}$	0 kNm/m	0 kNm/m
Uplift differential moment, $M_{hy.diff.o}$	0 kNm/m	0 kNm/m

If the caisson structure is closed, which is assumed at a water level of +0m CD, and the water level rises or falls at the seaside of the caisson structure due to the tide the water level differential increases, increasing loads on the sudden closure structure. The following hydrostatic loads are foreseen with maximum differential.

Table 3.16 Hydrostatic forces from wave – structure interaction (closed)

Hydrostatic loads horizontal	Low water -4m CD in front, +0m at rear	High water +6m CD in front, +0m at re
Hydrostatic force, $F_{hy.f}$	323 kN/m	1636 kN/m
Hydrostatic force, $F_{hy.r}$	-727 kN/m	-727 kN/m
Hydrostatic moment, $M_{hy.f}$	862 kNm/m	9818 kNm/m
Hydrostatic moment, $M_{hy.r}$	-2909 kNm/m	-2909 kNm/m
Uplift buoyancy (permanent), $F_{hy.buoy}$	-2424 kN/m	-3636 kN/m
Uplift differential hydrostatic (variable), $F_{hy.diff}$	-606 kN/m	-909 kN/m
Uplift buoyancy moment, $M_{hy.buoy.e}$	0 kNm/m	0 kNm/m
Uplift buoyancy moment, $M_{hy.buoy.o}$	36364 kNm/m	54547 kNm/m
Uplift differential moment, $M_{hy.diff.e}$	0 kNm/m	4546 kNm/m
Uplift differential moment, $M_{hy.diff.o}$	-3030 kNm/m	18182 kNm/m

A3.2 Stability of the concrete closure element - open

Reference is made to the design drawings in Volume 4.

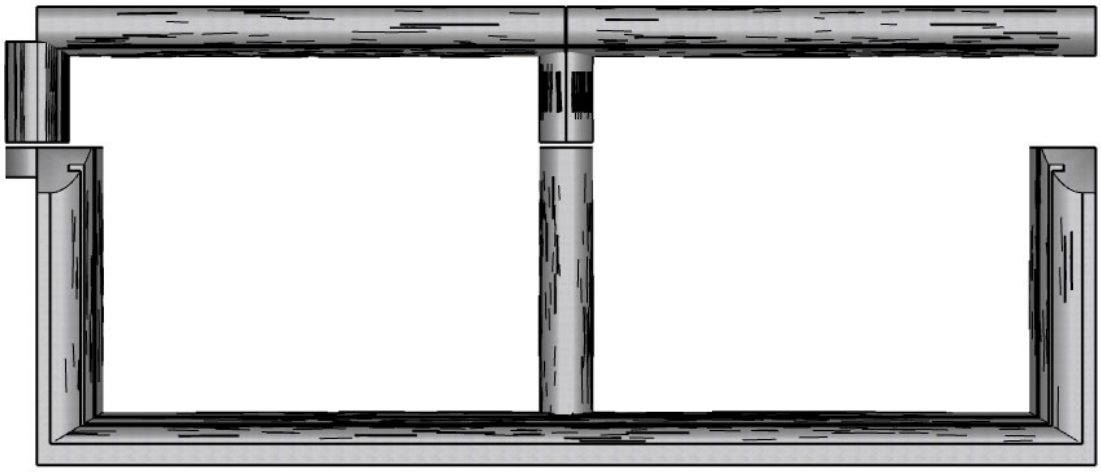


Figure 3.8 Front view of the concrete closure element

To more easily determine the wave loads on the structure a simplified caisson structure (with straight walls) is drawn, without the top structure. From this simplified structure the dead weight is determined.

The caisson is 100m long, as this was always the assumed length, and 30m wide. Two gates of 50m are foreseen. The floors and walls are 4m thick.

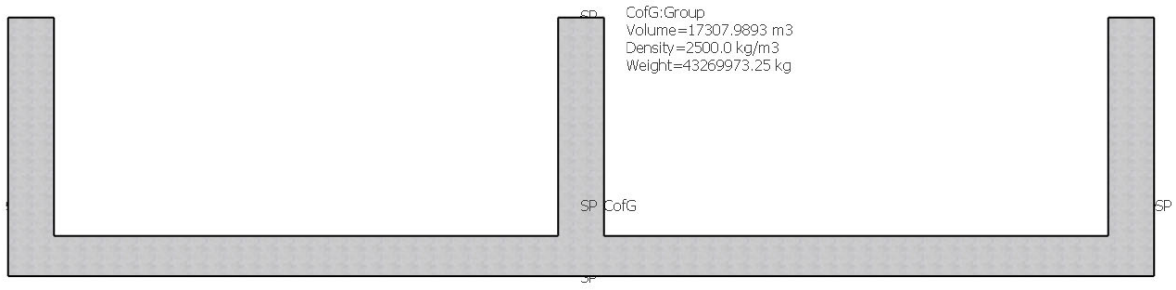


Figure 3.9 Schematised front view of the concrete closure element (side facing wave interaction)

The dead weight of the open structure without gates is 43,269,973 kg = 43,270 tonnes ≈ 424,478 kN. Once the structure gets hit by waves the horizontal wave force will mainly be on the vertical walls. Therefore, the total weight of the structure should be divided by the total width of the vertical walls, to determine the dead weight of the structure per meter length, which is the balancing force against the horizontal wave force per meter vertical wall.

The dead weight per meter of vertical wall becomes: F_{dw} = 35,373 kN/m

The stability of the structure is determined with the Mathcad sheet “stability of vertical composite breakwater/revetement, wave loading”. See the references of this sheet below.

1. Hydraulic Structures. Caissons, 2011. M.Z. Voorendt, W.F. Molenaar & K.G. Bezuyen
2. NEN 9997-1. Geotechnisch ontwerp van constructies - Deel 1: Algemene regels. 2017
3. Optimalisatie van een caisson golfbreker voor Maasvlakte 2, 1999. H.G. Voortman & J.K. Vrijling. Ministerie van Verkeer en Waterstaat
4. Probabilistic Design Tools for Vertical Breakwaters.Volume IIb Geotechnical aspects (PROVERBS). Balkema

FAILURE MODES CONSIDERED IN THIS SHEET:
1. Gapping
2. Overturning
3. Horizontal sliding

Overall stability (slip circle) is not included in this sheet. Further analysis on this failure mechanism shall be performed using specialised software e.g. D-stability.
Important notice: this sheet is only valid for caissons with a symmetric weight distribution.

The resulting unity checks are as follows:

Table 3.17 Unity checks wave – structure interaction (open)

Unity checks wave loading	Low water -4m CD	High water +6m CD
Unity check gapping, $UC_{gapping}$	0.11	0.13
Unity check overturning, $UC_{overturning}$	0.10	0.12
Unity check horizontal sliding, $UC_{sliding}$	0.18	0.16

With low water the assumed dead weight of the structure is more than sufficient to withstand the wave loading. The dead weight of the structure can be reduced to 120,000 kN with the same width of vertical wall (12 m in total for 100 m caisson), to keep the unity checks below 1. $UC_{sliding}$ is then just below 1.

With high water the assumed dead weight of the structure is more than sufficient to withstand the wave loading. The dead weight of the structure can be reduced to 144,000 kN with the same width of vertical wall (12 m in total for 100 m caisson), to keep the unity checks below 1. $UC_{sliding}$ is then just below 1. However, when the gates are closed the weight of the caisson structure should be heavier to withstand the horizontal wave pressure over the full length of caisson.

A3.3 Stability of the concrete closure element - closed

The closed caisson structure has the added weight of the gates but also the added wave loading to the closed gates. The 50 m wide gates are assumed to be 500,000 kg (Appendix A8 - A9). The dead weight of the closed structure then becomes= 43,270 tonnes + 1,000 tonnes = 44,270 ton. The dead weight per meter length caisson is: F_{dw} = 4,342 kN/m.

The dead weight per meter length of the caisson without top structure is too low to withstand a 1/100-year wave condition if the gates are closed. The horizontal wave force acts over the full length of the caisson and the balancing dead weight force is not sufficient if the caissons are not ballasted with reclamation material or extra weight from a top structure is added.

The dead weight force of the caisson should be minimal 12,000 kN/m to withstand a 1/100-year wave condition of 4.5 m H_s. This results in a total weight of 122,324 ton. The open caisson structure with the gates is 44,270 ton, so therefore an additional weight of 78,054 tonnes should be added to the caisson if the caisson without backfill material is to be stable during the 1/100-year wave condition.

If a top structure is added, as was shown in the first figure of the caisson, it will add about 29,495 tonnes to the weight of the structure. With a total weight of 44270 + 29495 = 73765 ton, the 1/1-year wave condition just results in a stable structure with a H_s = 1.82 m during high water level of CD +6 m. Which is the governing water level for the stability of the caisson structure. It is assumed in this situation that the water level at the back side is also CD +6 m. The unity checks are as follows:

Table 3.18 Unity checks wave – structure interaction (closed)

Unity checks wave loading 1/1yr wave, caisson closed with top structure	Low water -4m CD	High water +6m CD
Unity check gapping, $UC_{gapping}$	0.10	0.72
Unity check overturning, $UC_{overturning}$	0.13	0.54
Unity check horizontal sliding, $UC_{sliding}$	0.15	0.99

A4 GRADUAL CLOSURE – DESIGN OF ROCK DAM

A4.1 Stability equations - current

The horizontal and vertical gradual closure components are constructed by dumping rocks in the water. This dumping can occur horizontally (forward dumping from landside) or vertically (waterborne). The stones need to be large enough to lay stable on the bottom floor or on the crest and slopes of the progressive dam section (horizontally or vertically). In other words, the stones need to be large enough to not be washed away by the occurring current. The required stone size for the stones to be stable follows from simplified design criteria, fit for the present feasibility design.

For horizontal closure the stability of the rocks at the progressing dam face is determined by using:

$$\Delta D_{n50} = A \cdot {u_c}^2$$

with $\Delta = (\rho_s - \rho_w)/\rho_w$ the relative density of the rocks, D_{n50} $[m]$ the nominal diameter of the rocks and u_c [m/s] critical flow velocity of the rock i.e., the current velocity above the stones start to move. A is a stability factor defining the equation below:

$$A = \frac{0.64}{K_a \cdot \psi \cdot C^2}$$

with ψ a damage factor or Shields coefficient and C the Chézy coefficient $[m^{0.5}/s]$. K_a represents a correction factor for the slope dam head:

$$K_a = \sqrt{\cos(\alpha) \cdot \sqrt{1 - \frac{tan(\alpha)^2}{tan(\varphi)}}}$$

with α the slope of the dam head, and φ the natural slope of the material.

For vertical closure the equation [REF] for A and K_a is different for the stability of the rocks at the dam crest, namely:

$$A = \frac{0.36}{K_a \cdot \psi \cdot C^2}$$

$$K_a = \sqrt{\frac{sin(\varphi - \alpha)}{sin(\varphi)}}$$

A4.2 Dam volume

The required amount of rock for the rock dam follows from the cross-sectional area of the dam multiplied by the length of the dam (volume). The cross-sectional area is based on a trapezoidal shape, Figure *4.10*. A trapezoidal shape for the closure dam is chosen because it can follow the natural slope of the rocks and therefore no additional concrete elements are required. A construction road, required for the horizontal closure is also included on the crest. The dam in the model is subdivided into cells of $\Delta_x \; and \; \Delta_y$ The cross-sectional area of the cell is dependent on the slope of the trapezoid (α_d), the width of the crest (w_c) and the height of the trapezoid from the crest to the middle of the cell (h_c)

$$V_{damcell} = \Delta_x \Delta_y 2h_c \cos\alpha_d + w_c$$

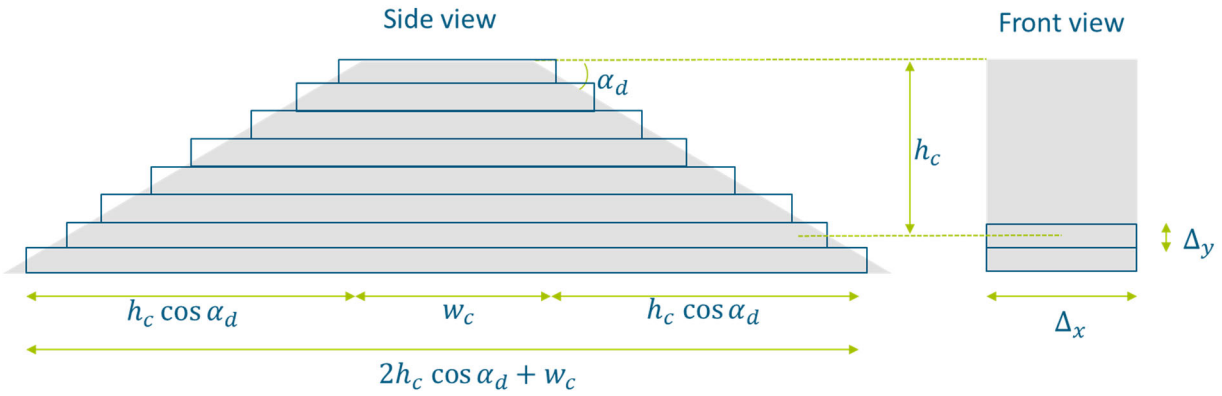


Figure 4.10 Determining the dam volume

A4.3 Shields coefficient

The stability of the rocks is determined by the Shields coefficient. The Shields coefficient describes the moment of acceptable movement of individual rocks, but the overall rock layer as a whole can still be considered stable. For example, if only 1 rock moves one would still say that the entire rocks dam is stable. On the other hand, if many rocks move the dam is not stable. The range of Shields coefficient varies between 0 [-] (no movement) and 1 [-] (complete transport of material. For a closure dam a Shields coefficient between 0.03 [-] and 0.04 [-] is commonly used.

A coefficient of 0.04 [-] or 0.03 [-] stems from small-scale experiments by Shields. Initially for sediment transport, but later also for small gravel to large rock. For gravel up to large stones the value of Shields proved constant (due to absence of viscosity effects by protrusion of the stones well above the viscous sublayer). For a value of 0.04 some incidental displacement can be expected. The value of 0.03 [-] is designated fully stable, by extrapolation from the experiments to zero transport. Whether a value of 0.03 [-] or 0.04 [-] should be used depends on the application. A value of 0.03 [-] should be applied to permanent structures and a value of 0.04 [-] can be used to assess the stability of a natural bed.

Said considerations apply to stability of the horizontal closure, stability of the vertical closure and stability of the bed protection. Design (hydraulic) conditions are time dependent, varying during a single tidal cycle as well as varying throughout the year between spring and neap tidal cycles. In addition, as the profile narrows, this directly affects the tidal range passing through the narrowed cross section and so maximum flow velocities acting on the rock structure in process of being built vary.

For horizontal closure it is considered too conservative to design all the rocks to be stable for the maximum velocity during a high spring tidal cycle. Hence a Shields factor of 0.04 [-] is considered more practical here for progressing dam construction. Dumping during the highest springtides normally will be interrupted and even more margin may be applied in the detailed design stage when some deformation of the progressing dam face is allowed. Additionally, for horizontal closure the front of the end dump is deformable, as the dam body is a massive body.

In the situation of a vertical closure a different Shields coefficient is used. This situation is more critical than the horizontal closure as regards rock instability. One reason is that the vertical dam build-up usually will be critical over a long length of the dam (as opposed to the end dump face exposure which is very locally). In

the ideal case of vertical build-up, the whole length of the dam crest is horizontal and can be critical. Hence, some damage will immediately result in damage over a long length of the dam. The same applies for uncertainties in flow prediction: whilst for end-dumping the flow structure is highly determined by the progressing dam face, for the vertical build-up the flow is more evenly distributed along the dam and some underestimate of the flow attack (e.g. by turbulent bursts progressing from the outer delta, which was observed at the Eastern Scheldt Storm Surge Barrier) will have a negative effect over a large length of the dam. Obviously, the dumped crest will have the capability to deform into a more rounded-off and elongated shape but this may give rise to a considerable delay in the progression and associated stone loss compared to the theoretical triangular profile. A more in-depth analysis can be done once the flow structure is well-understood and verified to field data in the earliest stages of the actual closure (the turbulent bursts have the greatest influence then). Additional wave attack should also be considered at that stage. A Shields coefficient of 0.03 [-] is therefore selected in this stage of the design.

For the bed protection a Shields factor of 0.03 [-] will be applied as well. For the top layer of the bed protection stones will be dumped at an under layer of finer stones or on a mattress. The exposure time of the bed protection will be significantly longer than for the stone made up of the rock dam. The attack by the current will be reduced when the opening is sufficiently far away for a specific part of the bed protection but this is the order of ~100 m till ~1000 m. Additionally, some damage can quickly develop into a large scour hole and can threaten the integrity of the closure dam. In a detailed design phase the required safety level for the bed protection should be further investigated. Often a higher stone class than required is used for the bed protection considering the overall threats, risks and safety.

A4.4 Chézy coefficient

The Chézy coefficient is determined by:

$$C = 18 \log_{10} \frac{12 \cdot d}{k_s} = 18 \log_{10} \frac{6\,d}{D_{n50}}$$

C	Chézy coefficient [$m^{1/2}/s$]
k_s	roughness height [m] ≈ 2 D_{n50}
d	water depth [m]
D_{n50}	nominal rock diameter [m]

For rocks a roughness height of $2 * D_{n50}$ is typically accepted for loosely dumped rock. The Chézy coefficient and the required stone size are dependent on each other and therefore the required stone size and the related Chézy coefficient should be solved iteratively.

The Chézy coefficient is largely dependent on water depth. The Chézy coefficient becomes very small with decreasing water depth compared to the rock size and very large with increasing water depth. For these cases a practical limit is assumed. From research it is know that the above equation for D_{n50} is limited to 5 times the water depth which limits C to around 25 $m^{1/2}/s$ on the lower end of the spectrum. For smaller depths the Chézy coefficient becomes smaller, but at the same time the Shields parameter increases. Roughly both effect balance and hence a C value is being taken also for extreme rough flow. This simplified approach is considered adequate for the present feasibility design (for detailed design the full approach for extreme rough flow should be taken.

At the high end of the spectrum, for very smooth flow at high water depths, C can be limited with increasing water depths to around 80 $m^{1/2}/s$, due to natural bed irregularities, form friction effects etcetera. This prevents underestimation of the rock diameter for smooth flow.

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A5 GRADUAL CLOSURE – DESIGN OF BED PROTECTION

A5.1 General

The function of using bed protection during a closure is to prevent structural instabilities in the direct surroundings of the closure structure due to scouring of the unprotected bed. The bed protection keeps the bed level around the dam structure at a constant level thereby preventing large scour holes near the structure. The requirements for the design of bed protection can be deduced from its three functions:

- **Sand tight:** This prevents the sand from moving and therefore a stable bed is guaranteed. To meet this requirement the protection can be executed in the form of a filter layer which is permeable for water but not for sediments. Another possibility is to form an impermeable layer. However, in this situation, water pressure differences will have to be dealt with. Sand tightness can be realized by applying subsequent layers of sedimentary material (from fine to coarse going upward) and/or by applying a geotextile that is being ballasted during or after installation.

- **Stability:** The top layer of the bed protection must be able to withstand the incoming forces of the flow, waves, and other occurring water forces. In the case of a closure dam, the flow velocity forms the normative force and is therefore the major driving force considered in the design of bed protection. In the equations also the water depth has an influence via the C-value.

- **Length:** The bed protection requires enough length to prevent possible instabilities in the soil generated by scour holes to approach the closure works. The occurrence of a scour hole can't be prevented, but the instability of the soil near the structure can be prevented with enough length of bed protection.

The cost of the bed protection is closely related to the total volume of the bed protection. The required length becomes an influential aspect of bed protection design which is related to the scour hole development behind the protection. Scour hole development is largely dependent on time and flow velocity. The required area of bed protection is found by multiplying the required bed protection length with the dam length.

Stability is guaranteed by choosing a stable rock D_{n50} of bed protection. Furthermore, it's assumed that the layer thickness is $2 \cdot D_{n50}$. The thickness is therefore mostly dependent on the maximum flow velocity occurring over the bed.

The volume bed protection including rock grading can be derived by multiplying the required thickness with the required area. The grading is derived by sorting the required volume over the available hydraulic rock gradings.

Bed protection is also required under de dam itself. The area of the dam footprint is taken up into the total area required since geotextiles might be necessary here. However, no stable rock layers are required here, because these are already included in the dam design. So this volume is excluded from the results.

A5.2 Bed protection length

The length of bed protection is directly related to the scour hole depth h_s via failure (collapsing) of the slope. Important factors are the slope before failure β and after β_f . The relation in its simplest and conservative form is given below (Figure *5.11*) which assumes $Y_d = 0$.

$$L_{Bed\ protection} = \frac{1}{2} Y_m (\cot \gamma_2 - \cot \beta_\alpha)$$

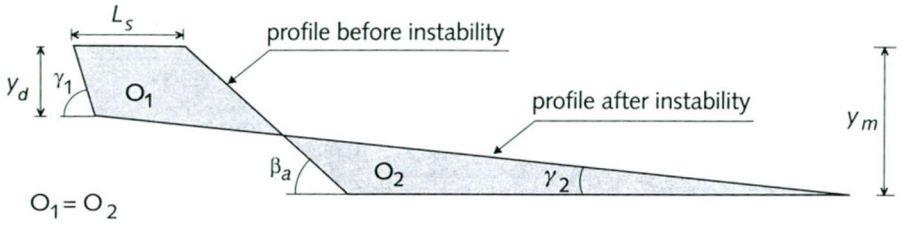


Figure 5.11 Relation bed protection length and scour hole depth (scour manual)

Y_m = Scour hole depth [m]
$\cot \beta_\alpha$ = Slope before failure [-] ~ assumed to be 4 [19]
$\cot \gamma_2$ = Slope after failure [-] ~ assumed to be 16 [19]

Table 8.19 Failure slope values before and after failure [REF]

storage parameters	flow slides and shear failures (Silvis 1988) Mean values	shear failures (Konter et al 1992), extreme values	flow slides (Konter et al 1992) Extreme values
$\cot \beta_\alpha$	3.7	4	4
$\cot \gamma_1$	2.7	2	2
$\cot \gamma_2$	16.3	8	20

A5.3 Scour hole depth

The scour hole depth is a time dependent process and is largely dependent on the occurring flow velocity. The scour depth is derived using Breuser's formula [20] for time dependent clear water scour:

$$y_m(t) = \frac{(\alpha_t U - U_c)^{1.7} h_0^{0.2}}{10\Delta^{0.7}} t^{0.4}$$

In which,
α_t turbulence factor [-]
U soliciting flow velocity over the bed [m/s]
Uc critical flow velocity bed material [m/s]
Δ relative density bed material [-]
h_0 water depth [m]
t time [hours]

The Breusers formula is developed for the active development stage of scour, but not for near-equilibrium situations. The formula is known to overestimate the scour depth for larger depths and near equilibrium. From measurements at the Eastern Scheldt, it is known that the maximum scour depth that can be reached is physically limited to around 2 times the maximum water depth. It should be noted that for Kalpasar a limit of 4 times the water depth is taken (as regards uncertainty about the validity of the limit found for the Eastern Scheldt).

19 Silvis (1988), Scour Manual OR
Hoffmans, Verheij (2021), "Scour Manual - Current-Related Erosion",
[20] *REF*

A5.4 Soliciting flow velocity

The soliciting flow velocity over the bed can be computed by using continuity under assumption that the reattachment point is always before the end of the bed protection and therefore the flow is in its active developing stage. Since the reattachment point is theoretically around 10 times the water depth and the maximum depth is 25m, the bed protection is assumed to always be longer than 250 m (which is the case at these water depths). For the present feasibility design we assume also that adaptation to a normal logarithmic flow profile has taken place (in reality this will also take some distance). For detailed design the actual flow adaptation after reattachment should be taken into account. For now the simplified equation reads:

$$U = U_0 \frac{h_{sill}}{h_0}$$

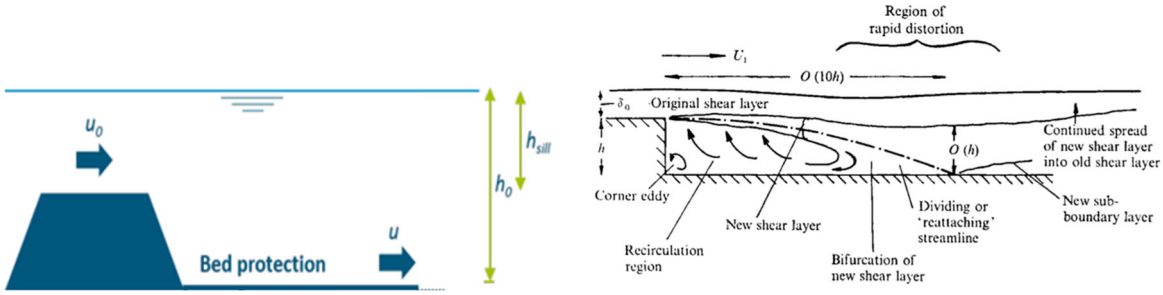


Figure 5.12 Soliciting flow velocity over bed protection (left) and flow pattern over a sill structure (right)

Furthermore, for tidal conditions, the flow is time-dependent u(t) and needs to be integrated for the tidal cycle in the scour formula. The flow has a maximum peak flow velocity (û) during the tidal cycles (for ebb flow or flood flow).

For simplicity reasons, the integration of the term with the time-dependent flow velocity u(t) can be replaced by using the maximum flow velocity during an ebb or flood tide û for an adaped time window of the ebb flow or flood flow cycle.

To this end, û should be multiplied with a factor β times the actual time window of ebb and flood flow. This factor is obviously smaller than 1,0. The factor is dependent on the form of the tidal curve (for a sinusoidal tidal wave the theoretical value is 0,78). In practice the value of β is in between 0.75 and 0.85. Here, the average value is taken: β = 0.80 which is accurate enough for feasibility design, as regards other uncertainties.

A5.5 Critical flow velocity

The critical flow velocity of the undisturbed bed is defined by the Shields relation for motion for fine sandy and cohesionless material on a flat bed.
This velocity reads:

$$U_c = C\sqrt{\Psi_c D_{n50} \Delta}$$

In which

Ψ_c = Shields parameter (assumed to be 0.03)
D_{n50} = Nominal sediment diameter (assumed to be 0.001 m)
C = Chézy coefficient

A5.6 Turbulence factor

The turbulence factor alpha is dependent on the closed section and the type of closure (horizontal or vertical). Measurements show increasing turbulence factors with increasing closing percentage. The turbulence factor is largest for horizontal closures, due to three-dimensional effects. Therefore, this value is used to have a conservative approach. Important is that turbulence diminishes with length of the bed protection. Since the scour hole depth is limited to $4h_0$ which limits L/h0 to around 24 with the chosen failure slopes, the turbulence factor will be relatively small for the maximum scour depth. Furthermore, 3D effects are present for a short time, but 2D effects dominate due to the long period of 2D flow. Therefore, the following practical approximation for the turbulence factor is taken.

$$\alpha = 2 \cdot \frac{A_c}{100} + 2$$

In which:
A_c = Cross sectional area closed [%]

With the high flow velocities during the closure, it is expected that the practical approximation will generally be realistic and not too conservative. In some cases, the factor might be understated however(during horizontal closures in low water depth). This approach is considered as sufficient for the present feasibility phase, but the method should be extended for detailed design.

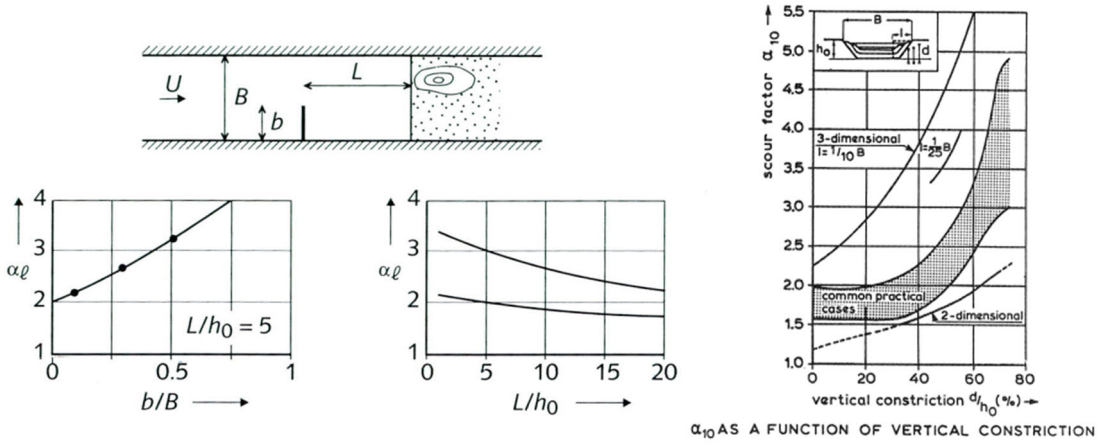


Figure 5.13 Turbulence factors for horizontal constriction (left) at L/h = 5 and vertical constriction (right) at L/h = 10

A6 PRODUCTION CAPACITY

A6.1 Introduction

The duration that it takes to close a cell is dependent on the construction capacity. To get an initial insight in the available construction capacity of a rock closure the following assumptions have been made, which is fit for present purpose:

- A horizontal closure is performed with dump trucks (land-based equipment)
- A vertical closure is performed with split barges (water-based + land-based equipment)

It is further assumed that the split barges can close the complete cross-sectional area. However, since they are limited by the water depth, they normally can only close the dam to a level of High water MSL -6.5 m or in this case around MSL -2.0 m.

Secondly, it is assumed that all rocks are stockpiled beforehand and are within 2 km of the dam start or port. In the following sections the potential horizontal and vertical construction capacities are determined.

A6.2 Horizontal closure capacity

Assumptions:

- Trucks will be deployed equally both sides of the gap.
- The dump trucks will be filled by front end loaders (1 m3) based at the stockpile
- Construction will take place for 22 hours out of 24, the remaining 2 hours per day being allocated for work force shift changes.
- On average one dump truck is expected to be discharging its load every 60 seconds on both sides of the gap. (This was every 30 seconds at the Saemangeum dam)
- Work will not be continued during monsoon period (3 months/year)
- Simultaneous to the dumps, reprofiling of the road will be done
- Gabions can be placed by end tipping from trucks in a similar way to that envisaged for the rock fill.
- The trucks have a loaded average speed of 20 km/h.
- The required crest width is 2.5 times the width of the truck used
- A truck requires 1.5 times its length to turn on the crest
- Enlarged bunds may be required every 500 meters to turn the trucks (Figure *6.14*)
- Overall inefficiency 5%: Downtime, shift changes go wrong, accidents

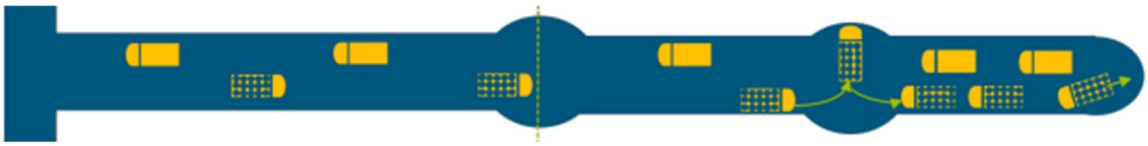


Figure 6.14 Logistics using large bunds for trucks to turn on the dam

Ref: Wallingford, H. R. (2005). Engineering review on the final closure of the Saemangeum Dike. Technical report, TU Delft.

Types of trucks
Below three types of trucks are discussed:

Rigid haul truck: Rigid haul trucks, also known as off-road trucks or rigid dump trucks, are a type of dump truck that are specifically designed for transporting rock and other materials in surface mining and quarrying

operations. They have an open bed with a hinged rear gate that can be raised to dump the load, and they are typically used for transporting materials over rough terrain. Rigid haul trucks can range in size from small, medium, to large and can have a maximum tonnage ranging from 50 to 450 tons.

https://en.wikipedia.org/wiki/Haul_truck

Articulated dump truck: Articulated dump trucks have a joint in the middle of the vehicle, allowing the front and rear halves to move independently. They are often used for transporting rock and other materials in rugged or off-road conditions. Articulated dump trucks can range in size from small, medium, to large and can have a maximum tonnage ranging from 25 to 40 tons.

https://www.volvoce.com/europe/en/products/articulated-haulers/#/all

On-road dump truck: On-road dump trucks are designed for use on paved roads and are typically used for transporting a variety of materials, including rock. They have an open bed with a hinged rear gate that can be raised to dump the load. On-road dump trucks can range in size from small, medium, to large and can have a maximum payload ranging from 10 to 30 tons. From contractors in India, it is known that small road trucks (10-tonne) are available.

https://constructionmentor.net/standard-on-road-dump-trucks/

In below, the specifics for all four dump trucks can be found.

Table 8.20 *Types of trucks*

truck type	maximum capacity [m3]	width [m]	length [m]	unload time [s]	turning radius [m]	turning width on enlarged bunds [m]	crest width required [m]	enlarged bunds required
small truck	6	2.3	6	60	9	9	5,75	yes
road trucks	18	2.5	8	60	12	12	6,25	yes
articulated truck	40	4	13	60	10	20	10	yes
rigid haul truck	300	10	21 m	60	31	31	25	yes

Number of trucks

The number of trucks that is used is a set number based on the assumption that every minute one dump truck will dump their load. The production capacity can only be increased by increasing the truck size or changing the frequency of the dumps.
The number of trucks required is dependent on the driving time from the stockpile to the bund and the loading time of a truck. Trucks can drive 20 km/h on the dam.
The minimum driving distance from the stockpile to the bund is assumed at 500 m which can be done in around 1 min (2 min round trip).
For a scenario without work islands and without caissons. The maximum distance to from the stockpile to the bund is 15 km (roundtrip 30 km) which takes around 1.5 hours.
The loading time using front end loaders is dependent on the capacity of the truck and the number of front-end loaders. For larger rigid haul trucks, loading will take a lot longer.
A front-end loader can load 1 m3 per minute. However, multiple front-end loaders can be used to load the trucks in case the truck capacity is very high. The assumption is that loading a truck can be done in 10 minutes.

For 40 m3 trucks, this means that 4 front end loaders need to continuously load the trucks. For 20 m3 trucks, only 2 front end loaders are required. It is assumed that a maximum of 4 front end loaders can load a truck at the same time.

Since every minute one truck will arrive at the loading spot. The number of front-end loaders is equal to 10 times the required number of loaders per truck.

truck capacity	loading time [min]	number of front end loaders per truck	total number of frond end loaders per bund
<10	10	1	10
>10 m3 and < 20 m3	10	2	20
>20 m3 and < 30 m3	10	3	30
>30 m3 and < 40 m3	10	4	40
>40 m3 and <100 m3	10-25	4	40
>100 m3 and <200 m3	25-50	4	40
>200 m3 and <300 m3	50-75	4	40

The required volume of front-end loaders per bund is the same for all scenarios. In a scenario with work islands the maximum distance is reduced by dividing the maximum distance by the number of work islands. However, the number of working fronts increases with two per work island. Furthermore, in a scenario with caissons, the total maximum driving distance is also reduced by the length of caisson.

The total number of trucks is defined for each truck size in the tables below. Since the number of trucks is also dependend on the scenario, the number of trucks is defined for the following scenarios:

- No work islands and no caissons (Table 8.21)
- Two work islands and 10 km caissons (table…)
- Two work islands and 15 km caissons (table…)
- Two work islands and 20 km caissons (table…)

For the scenario without work islands and caissons using regular truck sizes up to 40 m3, the minimum volume of trucks required is 24. To complete the last phase, 200 trucks are required. For the scenario's with work islands and caissons, a minimum of 72 trucks are required. Depending on the scenario a maximum of 90 to 120 trucks are required to complete the last phase.

Table 8.21 Scenario 1 and 3: No work islands and no concrete closure element

truck capacity	loading time [min]	minimum driving time roundtrip	maximum driving time roundtrip [min]	minimum total time [min]	maximum total time [min]	minimum number of trucks per bund	maximum number of trucks per bund	minimum total number of trucks	maximum total number of trucks
<10	10	2	90	12	100	12	100	24	200
>10 m3 and < 20 m3	10	2	90	12	100	12	100	24	200
>20 m3 and < 30 m3	10	2	90	12	100	12	100	24	200
>30 m3 and < 40 m3	10	2	90	12	100	12	100	24	200
>40 m3 and <100 m3	10-25	2	90	12	100-115	12	100-115	24	200-230
>100 m3 and <200 m3	25-50	2	90	12	115-140	12	115-140	24	230-280
>200 m3 and <300 m3	50-75	2	90	12	140-165	12	140-165	24	280-330

Table 8.22 Scenario 14: with 2 work islands using 10 km final closure gap

truck capacity	loading time [min]	minimum driving time roundtrip	maximum driving time roundtrip [min]	minimum total time [min]	maximum total time [min]	minimum number of trucks per bund	maximum number of trucks per bund	minimum total number of trucks	maximum total number of trucks
<10	10	2	10	12	20	12	20	72	120
>10 m3 and < 20 m3	10	2	10	12	20	12	20	72	120
>20 m3 and < 30 m3	10	2	10	12	20	12	20	72	120
>30 m3 and < 40 m3	10	2	10	12	20	12	20	72	120
>40 m3 and <100 m3	10-25	2	10	12	20-35	12	20-35	72	120-210
>100 m3 and <200 m3	25-50	2	10	12	35-60	12	35-60	72	210-360
>200 m3 and <300 m3	50-75	2	10	12	60-85	12	60-85	72	360-510

Table 8.23 Scenario 15: with 2 work islands using 15 km final closure gap
The number of front-end loaders per bund is the same

truck capacity	loading time [min]	minimum driving time roundtrip	maximum driving time roundtrip [min]	minimum total time [min]	maximum total time [min]	minimum number of trucks per bund	maximum number of trucks per bund	minimum total number of trucks	maximum total number of trucks
<10	10	2	7.5 ~8	12	18	12	18	72	108
>10 m3 and < 20 m3	10	2	8	12	18	12	18	72	108
>20 m3 and < 30 m3	10	2	8	12	18	12	18	72	108
>30 m3 and < 40 m3	10	2	8	12	18	12	18	72	108
>40 m3 and <100 m3	10-25	2	8	12	18-33	12	18-33	72	108- 198
>100 m3 and <200 m3	25-50	2	8	12	33-58	12	33-58	72	198-348
>200 m3 and <300 m3	50-75	2	8	12	58-83	12	58-83	72	348-498

Table 8.24 Scenario 16: with 2 work islands using 20 km final closure gap
The number of front-end loaders per bund is the same

truck capacity	loading time [min]	minimum driving time roundtrip	maximum driving time roundtrip [min]	minimum total time [min]	maximum total time [min]	minimum number of trucks per bund	maximum number of trucks per bund	minimum total number of trucks	maximum total number of trucks
<10	10	2	5	12	15	12	15	72	90
>10 m3 and < 20 m3	10	2	5	12	15	12	15	72	90
>20 m3 and < 30 m3	10	2	5	12	15	12	15	72	90
>30 m3 and < 40 m3	10	2	5	12	15	12	15	72	90
>40 m3 and <100 m3	10-25	2	5	12	15-30	12	15-30	72	90-180

truck capacity	loading time [min]	minimum driving time roundtrip [min]	maximum driving time roundtrip [min]	minimum total time [min]	maximum total time [min]	minimum number of trucks per bund	maximum number of trucks per bund	minimum total number of trucks	maximum total number of trucks
>100 m3 and <200 m3	25-50	2	5	12	30-55	12	30-55	72	180-330
>200 m3 and <300 m3	50-75	2	5	12	55-80	12	55-80	72	330-480

Table 8.25 Key logistical (land-based) parameters used

truck capacity	loading time	number of front end loaders per truck	total number of frond end loaders at one loading point
<10	10	1	10
>10 m3 and < 20 m3	10	2	20
>20 m3 and < 30 m3	10	3	30
>30 m3 and < 40 m3	10	4	40
>40 m3 and <100 m3	10-25	4	40
>100 m3 and <200 m3	25-50	4	40
>200 m3 and <300 m3	50-75	4	40

Resulting horizontal capacity

The following equation is used to determine the horizontal construction capacity

$$C_{horizontal} = C_{truck\ (m3)} \cdot \frac{3600}{T_{dump_{seconds}}} \cdot 24 \cdot \frac{eff_{\%}}{100} \cdot \frac{12 - T_{monsoon\ (months)}}{12}$$

A6.3 Vertical closure capacity

The vertical capacity is dependent on the number of ships that can be utilized. Assume 3 of the larger contractors with split barges working together:

- Jan de Nul (17 usable ships with a total of 51.000 m^3 capacity (maximum of 3,700 m^3 per ship)
- Van Oord (4 usable split barges, one side dumping vessel, total capacity 13,500 m^3)
- Rohde-Nielsen (6 split barges, total capacity 7.500 m^3)

Ref: https://www.vanoord.com/en/equipment/hopper-barge-and-pushbuster/
Ref: https://rohde-nielsen.com/fleet/hopper-barges
Ref: https://www.jandenul.com/fleet/split-hopper-barges

Assumptions

- Ship capacity 4.000 m3
- A dumping ship can dump twice a day during High Water
- Maximum sailing distance is 20 km (roundtrip 40 km), Speed: 20 km/hour (roundtrip 2 hours)
- Berthing and getting in position 1 hour
- Per ship one birth is required (long loading times vs dumping times)
- Loading time left by conveyer or crane: 8 hours, loading rate required: 500 m^3/hour
- Ship dumping can only be done until minimum high-water level – 6.5 m (draught ship)
- Distance from birth to stockpile (~2 km). Driving 2.5 minutes, roundtrip of 5 minutes
- Loading time trucks is 1 minute per m^3 using front end loaders (1 m^3)

- Using smallest dump trucks (6 m^3 or 18 m^3) available to load ships from stockpile
    - For the 6 m^3 dump truck this results in 5 dumps per truck per hour
    - For the 18 m^3 dump truck this results in 2 dumps per truck per hour
- Overall inefficiency 10%: Downtime, shift changes go wrong, accidents

Per berth the following is required (cost per ship)
- A dredged access channel (300 m x 50m x 10 m)
- A dock with length of 130 m (length ship is 100 m)
- Three heavy duty feeders with conveyer belts with a capacity of 180 m^3/hour
- One or two cranes to support the conveyer belts with larger rocks.
- Three front end loaders at stockpile (1 m^3/s)
- 18 trucks (6 m^3) or 6 trucks (18 m^3)

Per ship, one birth is required since most of the time the ship is loading at this birth. Using only one high water a day to dump is inefficient. Although less loading equipment is required, the number of births and dredged channels stay the same. Therefore, if a ship is used, it should dump twice a day at high water to optimize the cost per tonnes placed material per birth.

Resulting vertical capacity:
The resulting capacity is dependent on the number of ships used. In this design we take up a range from 2 to 12 ships. 12 ships is 80% of the fleet capacity for both Jan de Null and van Oord. The probability that these ships are available is already quite small, so we have decided to set this number as maximally feasible. The resulting capacity follows from the following equation:

$$C_{vertical} = C_{ship} \cdot n_{ships} \cdot \frac{eff_{\%}}{100} \cdot \frac{12 - t_{monsoon}}{12}$$

Table 8.26 Number of ships, resulting daily capacity and required contractor fleet capacity.

number of ships	capacity [m^3]	daily capacity [m^3/day] at full efficiency	required contractor fleet capacity
2 ships	8,000	16,000	~20 % Jan de Nul
6 ships	24,000	48,000	50 % Jan de Nul
12 ships	48,000	96,000	80% Jan de Nul 80% van Oord
18 ships	72,000	144,000	100% Jan de Nul, 100% van Oord, 100% Rohde Nielsen

A7 CONSTRUCTION AND LOGISTICAL HUBS

A7.1 Introduction

Constructing work islands on the tidal flats can increase construction capacity during the closure. Stockpiles with rock can be build up on the islands during site preparation. The upside of these work islands is that the total closing time can be reduced. To evaluate how these work island will look like, a very conceptual design is sketched here.

A7.2 Design

Size

the size of the work islands is mostly based upon the required stockpile on these islands. The maximum required rock volume for the closure dam is around 80.000.000 m3 of rock. A stockpile is assumed to have a maximum height of 60 m. If this rock is distributed on two islands (each 500 m in width) from which 1/3 of the stockpile needs to be distributed, the required size of these islands needs to be around 900 meters (Table *8.27*). Furthermore, a port is required and some working space, so the work islands will be 1000 meters long.

Table 8.27 Work island design based on required stock pile volumes

distribution volume closure dam		volume [Mm^3]	stockpile area [Mm^2]	width of island [m]	length of island [m]
stockpile western shore	1/6	13.3	0.2		
stockpile eastern shore	1/6	13.3	0.2		
stockpile work island 1	1/3	26.7	0.4	500	900
stockpile work island 2	1/3	26.7	0.4	500	900
total	1	80			

Material

The islands will be build using a rock dam enclosure with a sand filling. The rock dam will have a slope of 1:2 and are on average 10 meters high. The required rock volume is around 600.000 m3 per island. The sandfill is around 5.000.000 m3 per island. The stockpile on the islands can be built during site preparation using ships, cranes and dump trucks as described in section A6.3

Effect on construction capacity

The simple effect on the construction capacity is that the horizontal closure capacity triples.

A7.3 Design approach

The previous sections provided fundamental insight in the design equations and the background information on the production capacity. It did not explain how these relations and numbers will be used for a closing scenario. This section provides this explanation. First the design approach is explained using a flow diagram and a example. Note, the example is chosen to explain the procedure and not the most realistic scenario.

In Figure *7.15* the design approach for single closing scenario is visualized. The design steps are:

1. A closing scenario is chosen. In the example a closing strategy using 10 km of open structure with a height of 20 m is selected. In Figure *7.16* the closing strategy is presented. The figure shows a vertical closure from – 24 m MSL to –19 m MSL followed by a horizontal closure of 7.5 km. The rest is filled up vertically to + 10 m MSL.
2. The cross-section of the closure dam is discretized considering the 10 km of sudden closure structure, see Figure *7.17*. The discretization in the horizontal is 250 m and in the vertical 1 m. The

discretization of 250 m is chosen because the flow velocities are known every 500 m of horizontal closure. Therefore, a closure step of 500 m means that 1 cell on the west and 1 cell on the east side is closed. The cell in the middle is 330 m rather than 250 m because the cross-sectional length is not dividable by 250.

3. Based on the closing scenario a closure step, horizontal or vertical, is selected.
4. By using the results of Volume 1 (REF) the current velocity and water level can be selected.
5. For the rock dam the is it important to know which cells will be closed. This is determined based on the closure step.
6. For the rock dam the required stone size (D_{n50}) to be stable is calculated using the water depth, the type of closure step and the current velocity.
7. For the bed protection, the open cells or water columns are required (differentiate only in x direction). In these columns (125), erosion of the bed will continue. To determine the open columns each phase, step 5 is inverted.
8. Each column that is open in time will suffer from erosion. the erosion rate is determined using the flow velocity over the dam and water depth which leads to the soliciting flow velocity at the end of the bed protection. Using the critical flow velocity (shields), the erosion rate is determined.
9. Each phase has a duration. The duration of each phase is based upon the volume that is placed in each phase. This depends on which parts of the dam are closed in which phase and the construction capacity (horizontal and vertical).
10. The next step is to determine the depth of the scour hole for each section using Breuer's equation for clear water scour.
11. The described loop continues until all the cells are closed. For the rock dam one could summarize these steps by plotting them on the grid of the rock dam.
12. A cell is closed in a specific phase, with a specific closing method under a specific velocity and water level, see Figure *7.18* until Figure *7.21*.
13. In Step 6 is required rock size is already determined for the rock dam. In Step 13 these are sorted in standard gradings. Figure *7.23* depicts the sorting in. When the stone grading becomes larger than 10-15 ton, it is labelled as concrete. Larger rocks than 10-15 tonnes is no longer a standard grading and concrete units need to be used. When the dam is above water the smallest stone size is selected, in this case 10-60 kg. Other materials, such as sand can also be used.
14. In Figure *7.24* the volume of each cell of the closure dam is depicted. The vertical line in the middle is because in Step 2 all cells are 250 in width except 1 column. Those cells have a width of 330 m. By coming the volume of each cell with stone grading in each cell a grading curve for the rock dam is determined, Figure *7.25*.
15. For the bed protection, the maximum depth of the scour hole per section is determined. This can be either the result of the time dependent scour process or it might have reached the equilibrium depth equal to four times the water depth.
16. The length of the bed protection is dependent on the failure slope if the slope between the scour hole and the dam collapses. This is roughly a factor 6 of the scour hole depth. See section A5.2
17. During the closure the bed protection needs to be stable. This means that it should withstand the maximum soliciting flow velocity over the bed. This flow velocity is also used to determine the scour depth (see section A5.3). The required stone size is determined using Shields equation for motion on flat beds.
18. The bed protection layer thickness is set to be $2D_{n50}$. Per column one thickness is determined. The required area is the bed protection length times the column length (250 meters) and the required volume per column is the required area times the layer thickness. The design D_{n50} per column is used to sort each strip of bed protection into a hydraulic rock grading type ranging from 10-60 kg to 10-15 tonnes rocks. A rock grading curve can then be produced.
19. By combining the grading curve for the rock dam (Step 14) and the grading curve for the bed protection (Step 18) the a grading curve for the entire rock structure is determined.

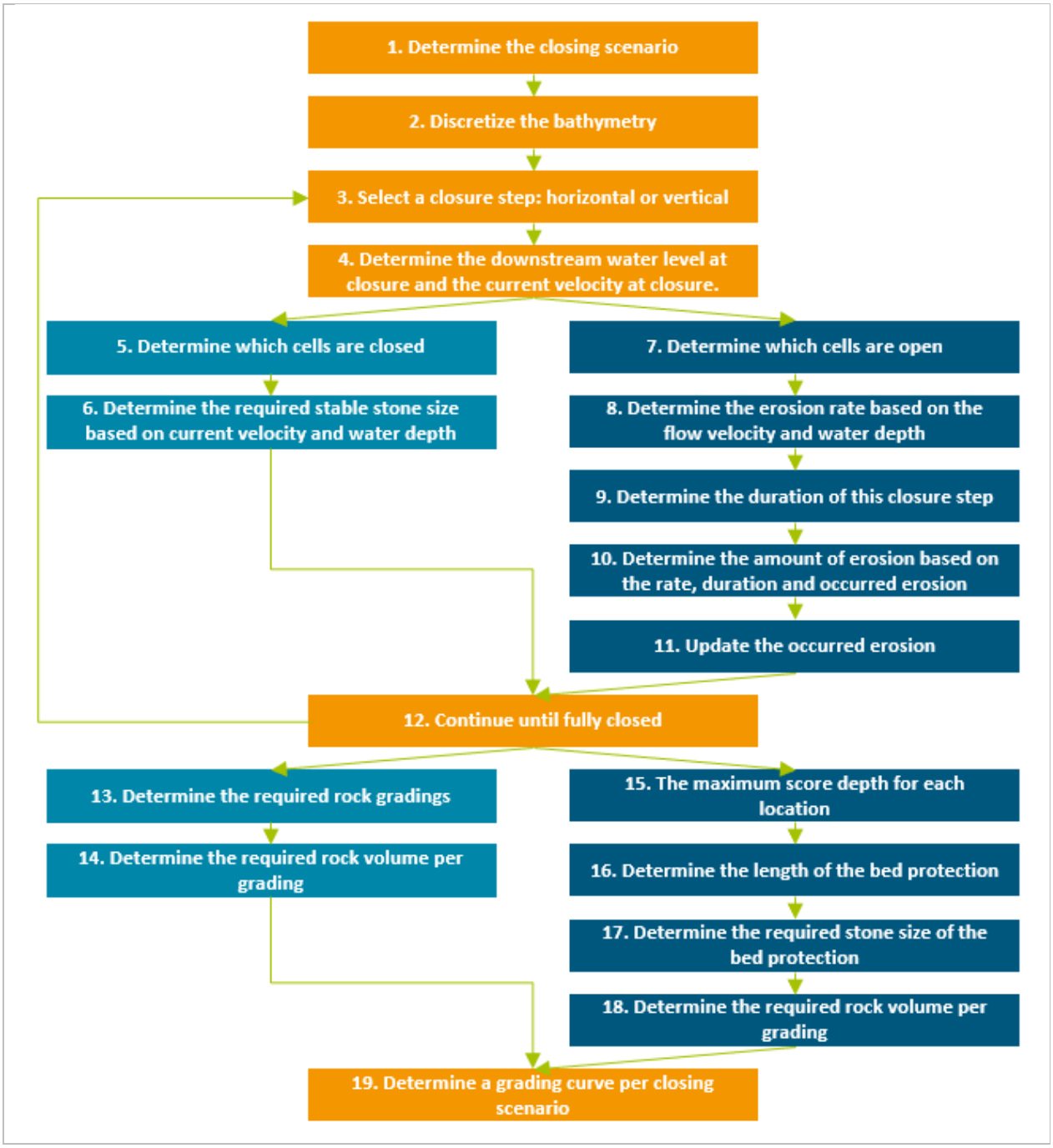


Figure 7.15 The design approach used for the rock structure.
The orange boxes are applicable to the rock dam and the bed protection, the light blue only applicable to the rock dam and the dark blue only applicable to the bed protection.

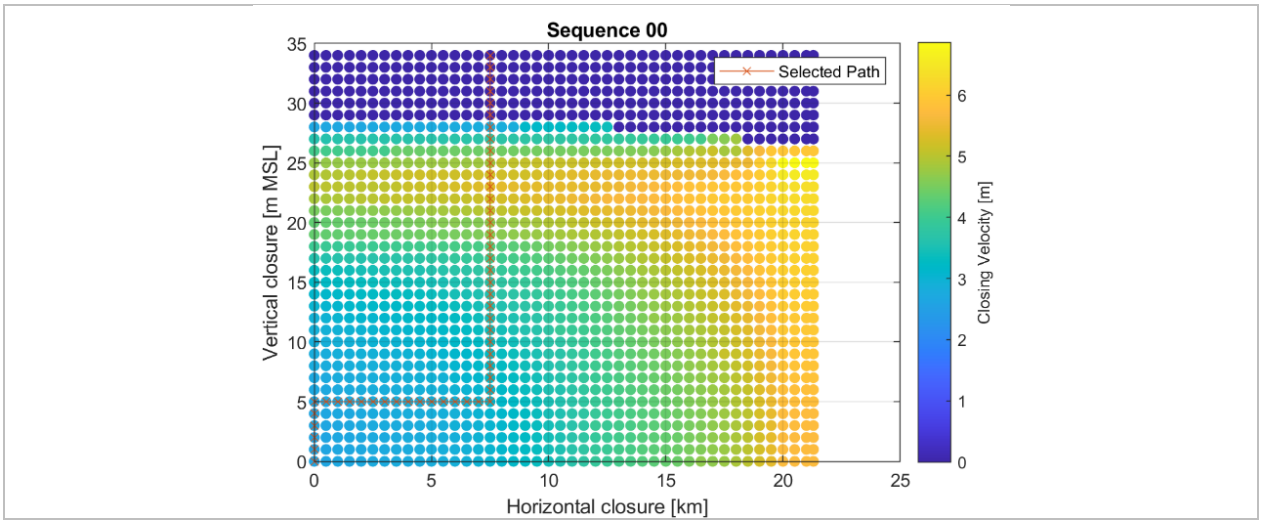


Figure 7.16 An example closing scenario of with 10 km of sudden closure structure
For the derivation of the flow velocities, reference is made to Volume 1.

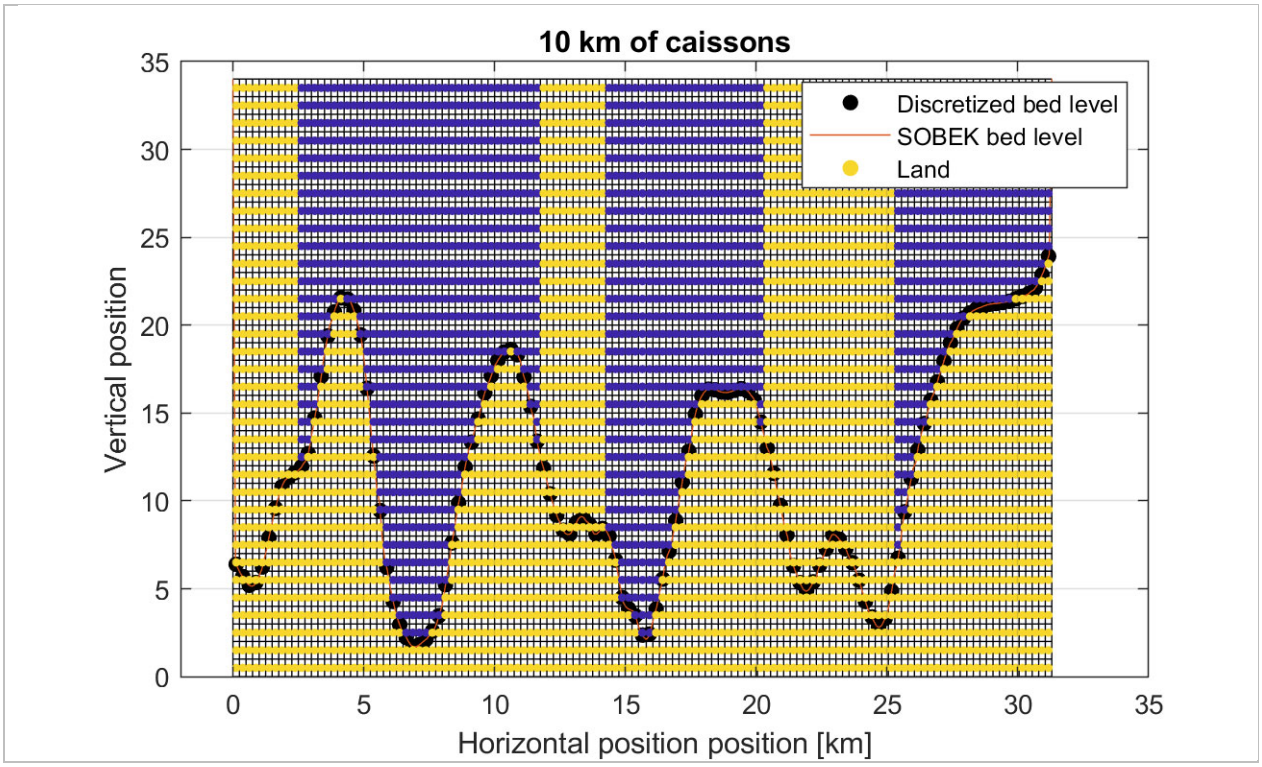


Figure 7.17 The discretised closure for a 10 km sudden closure structure
The blue dots indicate the cells which need to be closed. The yellow dots indicate cell which are not part of the rock structure because it is bed material, or a sudden closure structure is located here. The black dots indicate discretized bed level.

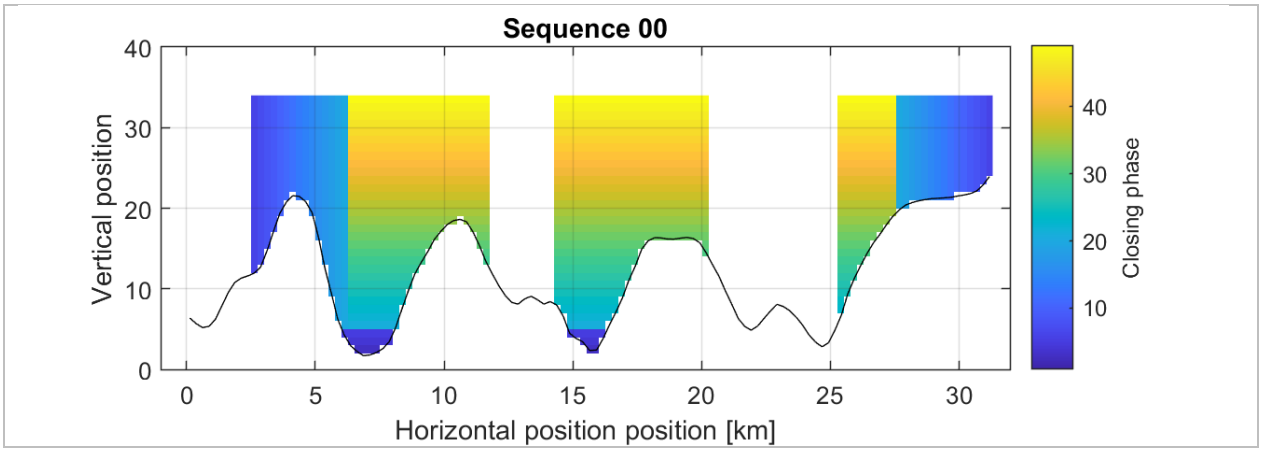


Figure 7.18 The closing phase of each cell for the example scenario.

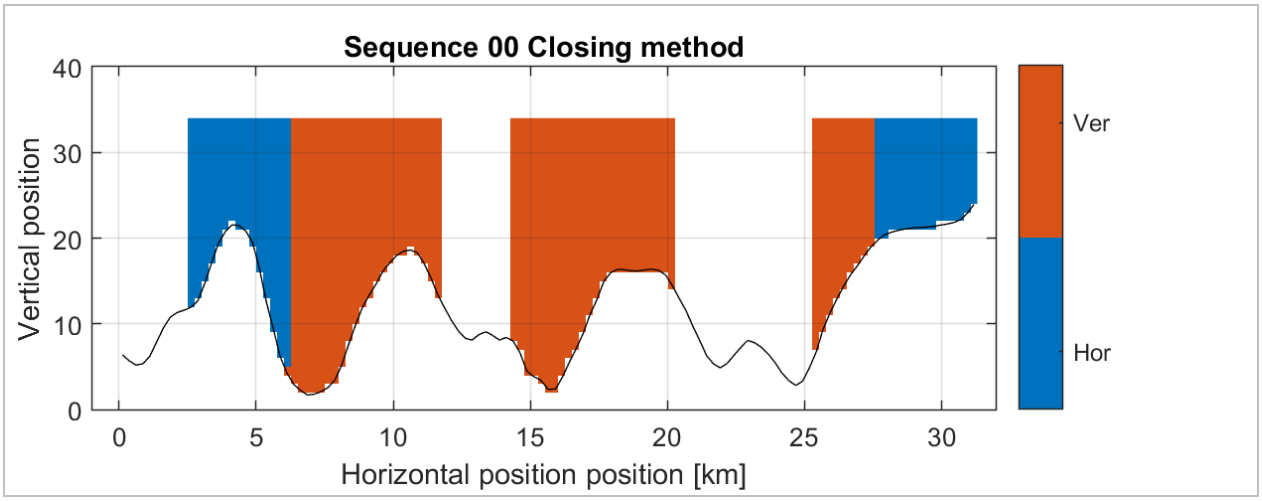


Figure 7.19 The closing method of each cell for the example scenario.
The white bars are not part of the rock dam because the sudden closure structure will be placed here.

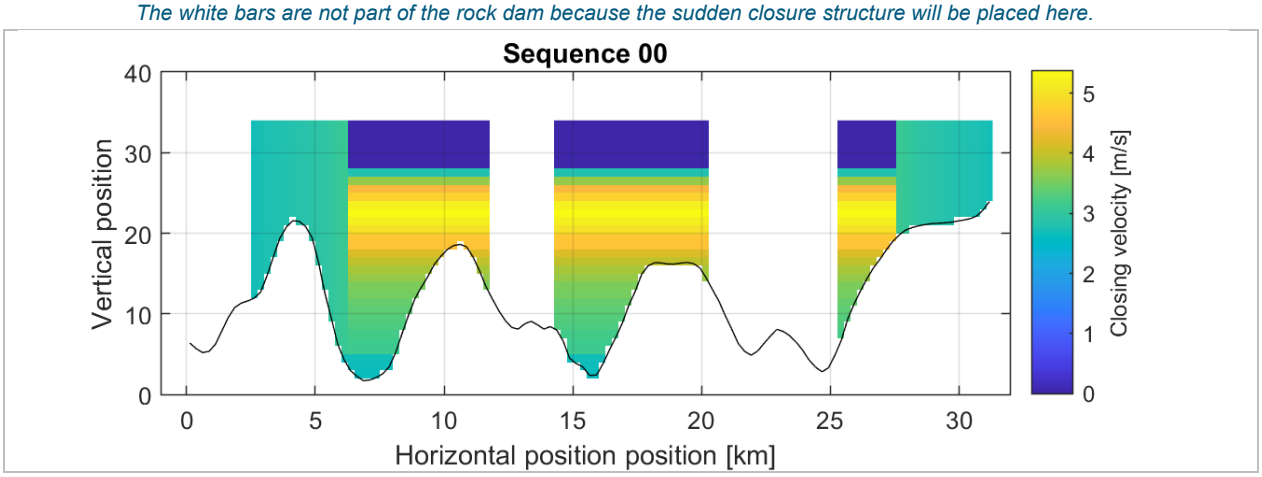


Figure 7.20 The closing velocity for each cell of Sequence 00.
The white bars are not part of the rock dam because the sudden closure structure will be placed here.

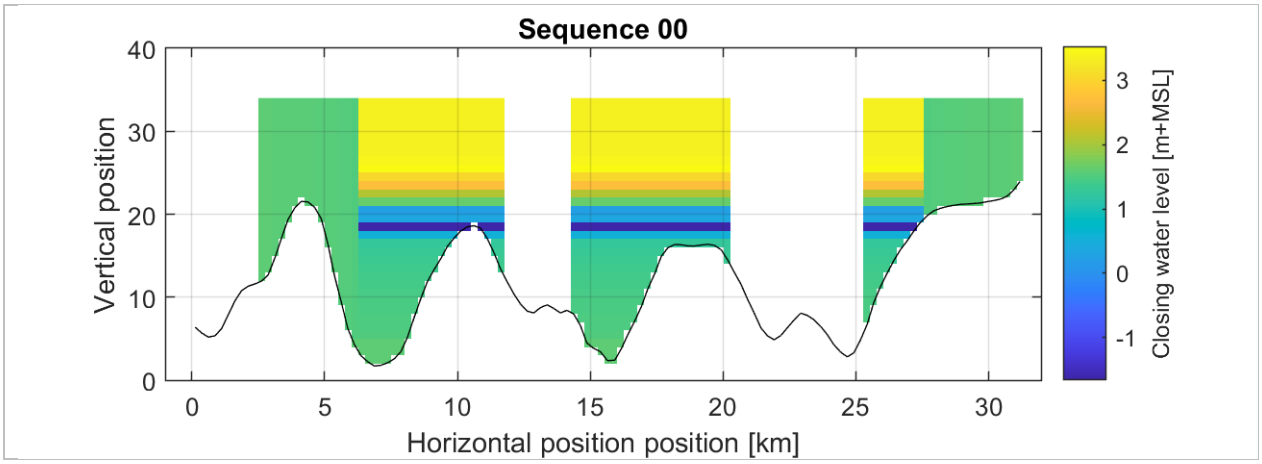


Figure 7.21 The closing water level for each cell of Sequence 00.
The white bars are not part of the rock dam because the sudden closure structure will be placed here.

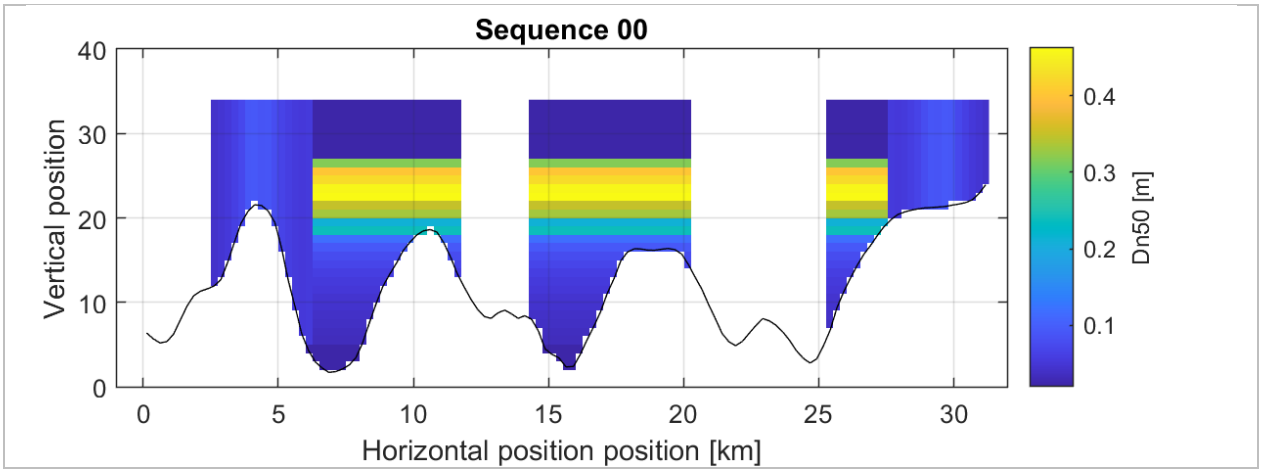


Figure 7.22 The required stone size to be stable for each cell of Sequence 00.
The white bars are not part of the rock dam because the sudden closure structure will be placed here.

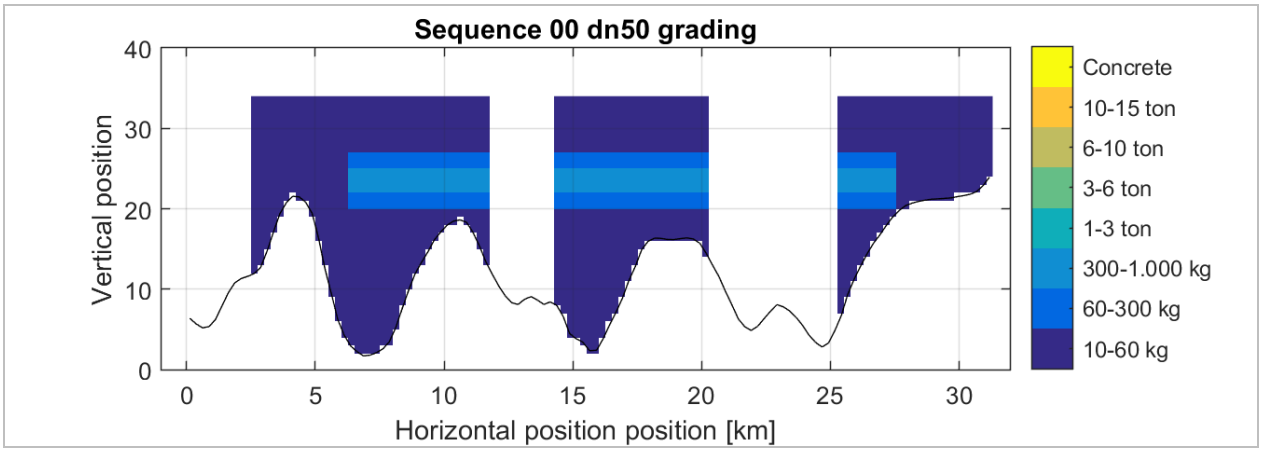


Figure 7.23 The stone size, sorted in standard stone gradings.
The white bars are not part of the rock dam because the sudden closure structure will be placed here.

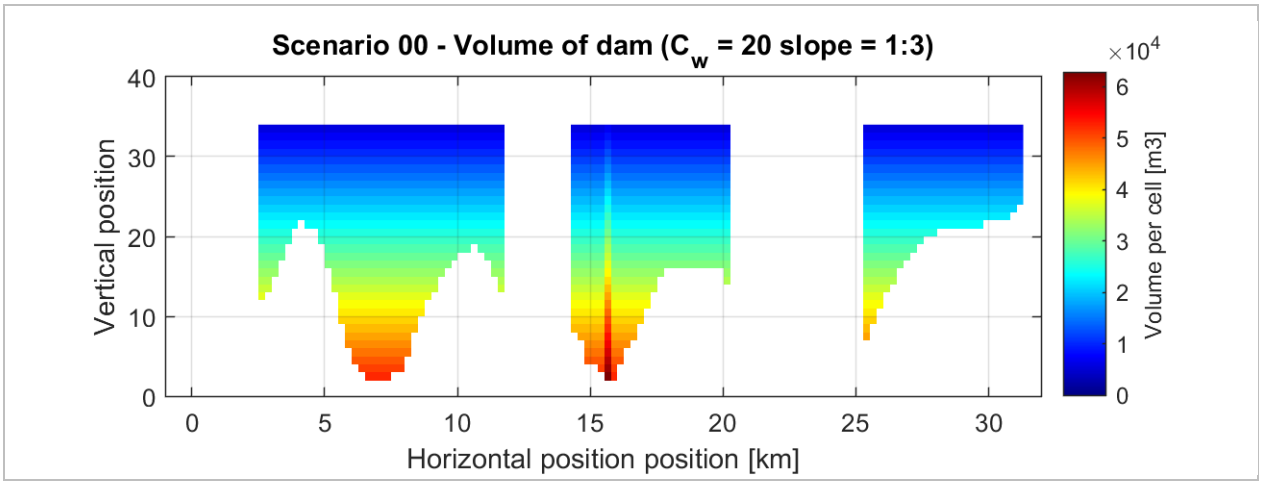


Figure 7.24 The volume per cell of the rock dam for a slope of 1:3 and a width of the crest of 20 m

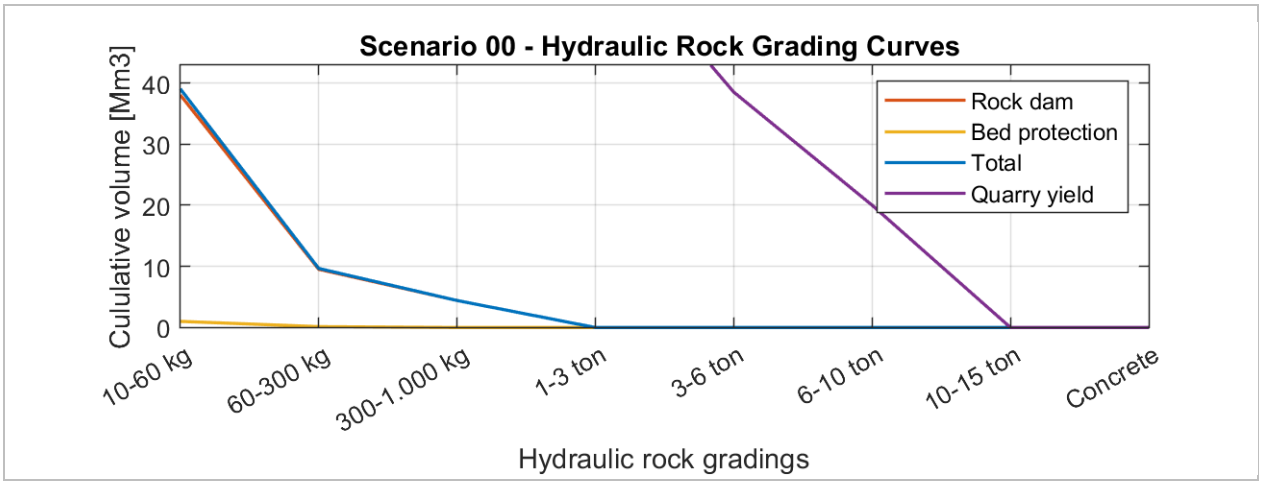


Figure 7.25 The grading curve for the rock dam, bed protection and the total rock structure for closing Scenario 00.

A8 TECHNICAL EVALUATION OF GATE TYPES

A8.1 Alternative A: vertical lift gates

The main loads on the operation mechanism of the vertical lift gates will be the weight of the gate and the friction when closing under hydraulic pressure on the gate. The friction when closing can be minimized by using rolling wheels at the gate. For now, sliding strips are used to guide the gate which give a higher friction but will be more suitable for the occasional use of the gate.

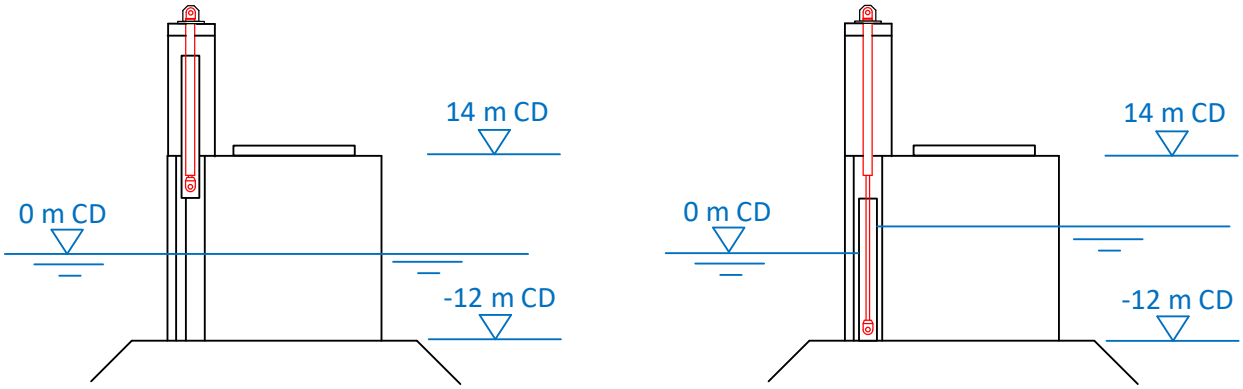


Figure 8.26 Vertical lift gate (20 m high)

A8.1.1 Operation loads

For the vertical lift gates the operation loads are determined. Given the weight of the gate, the hydraulic pressure and a friction coefficient of 0.2 [-], the results are shown in the figure below.

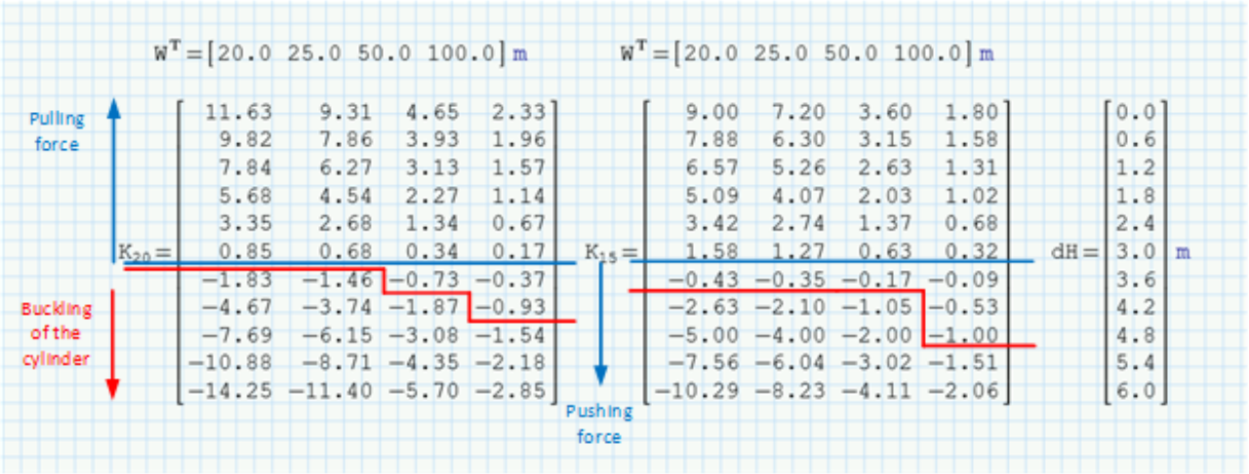


Figure 8.27 Results vertical lift gates

The table K_{20} gives the results of the 20 m high gate, the table K_{15} of the 15 m high gate. The columns present the results for the varying gate widths, the rows the difference in water level across the gate, with 0 m CD at the side of low water level.

The tables are dimensionless and present the quotient between the resulting operation force and the buckling load of the cylinders. When positive the gate will close under its own weight, when negative the gate must be pushed down with an operation mechanism. The tilting point pulling/pushing is shown with the blue line. When the gate closes under its own weight (above the blue line) a cable system could be used to operate the gate.

In the tables a red line is also shown. The red line shows the boundary at which buckling of the cylinder must be considered. For buckling it is assumed that the ring area of the piston is half the bore of the cylinder. Furthermore, the cylinders are dimensioned at a maximum working pressure of 200 bar. For the buckling length, an approximate length is used for the distance between the rod eye and bottom eye of the cylinder.

Please note that the results are based on indicative calculations to provide some insight. Exact calculations should be made in the further design.

A8.1.2 Cylinders

Cylinders are produced in standardized construction sizes, and accurate calculations must be made to determine the cylinder size required. For an indication, theoretical cylinder sizes for the gates are presented in the figure below, these are based on a maximum pressure in the cylinder of 200 bar, while holding the gates in open position and a ring area of the piston, half the bore of the cylinder.

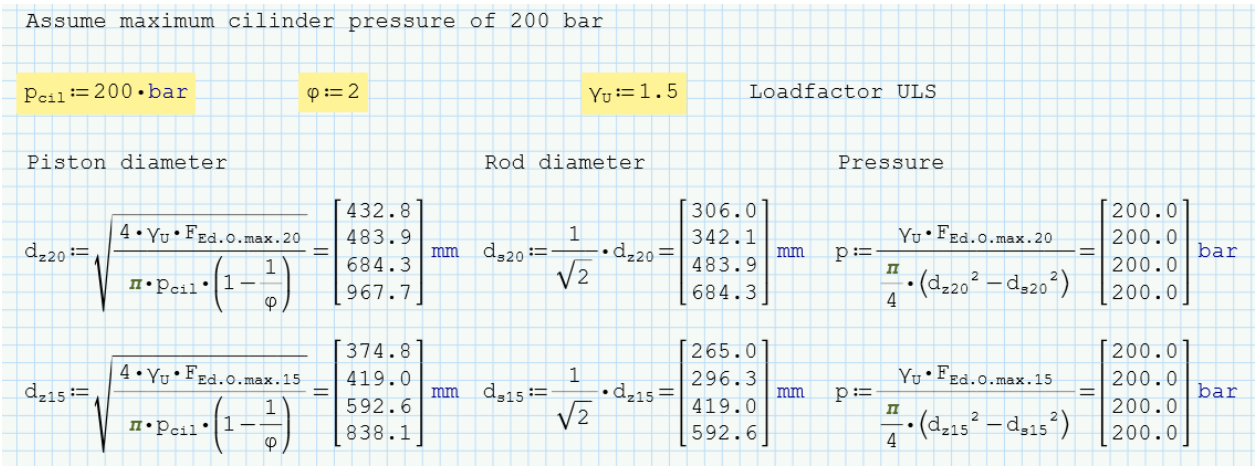


Figure 8.28 Theoretical cylinder dimensions

When using standardized cylinder dimensions, it would result in in the dimensions as shown table below.

Table 8.28 Standardised cylinders

	Width of the gate							
	20 m		25 m		50 m		100 m	
	bore	rod	bore	rod	bore	rod	bore	rod
Height	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
20 m	450	320	500	360	700	500	1000	700
15 m	400	280	450	320	600	420	850	600

A8.1.3 Hydraulic Power Unit

Up to a water level difference of 3 m, with at the low water level side 0 m CD, the gates will close under their own weight by simple discharge the hydraulic oil from the pressure side of the cylinders. To prevent the gates from locking (sticky drawer effect), the oil discharge from the cylinders must be controlled with a proportional valve so the gate closes evenly on both sides. Therefore, the cylinders, or the gate near the cylinders, must be equipped with a measuring system to extend the cylinders synchronously.

For testing purposes, the Hydraulic Power Unit (HPU) should be provided with pumps in order to open a gate after it has been closed. When using pumps while opening the gates there will be a risk of buckling of the cylinders due to the length of the cylinders.

Testing of the gates will be necessary to prevent the operation mechanism and the gates to jam and to detect non-noticeable failure of the operation mechanism during the building of the dam.

A8.1.4 Wire ropes

Up to a water level difference of approx. 3 m the gates can be closed by their own weight and a wire rope system could be used for the closure. When calculating the number of wire ropes per side of the gate, using a cable with a 60 mm diameter and a tensile strength of 1960 MP following results are obtained.

Table 8.29 Number of ropes

	Width of the gate			
	20 m	25 m	50 m	100 m
Height	ropes	ropes	ropes	ropes
20 m	4	4	8	14
15 m	4	4	6	12

For 60 mm ropes/cables a cable drum with a minimum diameter of 1120 mm will be necessary.

Assume a wire rope will be used

$k_{b.o} := 3.55$ Service factor wire ropes for ULS (K2 NEN3508)

$$F_{rope.o.20} := k_{b.o} \cdot F_{Ed.o.max.20} = \begin{bmatrix} 3481.4 \\ 4351.7 \\ 8703.4 \\ 17406.8 \end{bmatrix} kN \qquad F_{rope.o.15} := k_{b.o} \cdot F_{Ed.o.max.15} = \begin{bmatrix} 2611.0 \\ 3263.8 \\ 6527.6 \\ 13055.1 \end{bmatrix} kN$$

Assume steel wire rope with maximum diameter 60 mm and tensile strength 1960 MPa

$F_{br} := 2500 \cdot kN$ Minimum breaking force

Number of wire ropes per side of the gate

$$n_{rope.20} := ceil\left(\frac{F_{rope.o.20}}{F_{br}}\right) = \begin{bmatrix} 2.0 \\ 2.0 \\ 4.0 \\ 7.0 \end{bmatrix} \qquad n_{rope.15} := ceil\left(\frac{F_{rope.o.15}}{F_{br}}\right) = \begin{bmatrix} 2.0 \\ 2.0 \\ 3.0 \\ 6.0 \end{bmatrix}$$

Figure 8.29 Number of wire ropes

Winches are needed to handle the wire ropes. To operate the winch a drive mechanism is needed with among others a gearbox, a brake system, motor, variable speed drive (suitable for 4-quadrant operation) and a braking resistance to absorb the released energy during motor-braking when lowering the gate. For a controlled motion of the gate with the wire rope system a measuring system is needed to control a synchronous motion of both sides of the gate. For testing purposes, the winches can be used to re-open the gate.

A8.1.5 Considerations

Some considerations are made with respect to the operation of the gates using either hydraulic cylinders or wire rope systems. More types of operation systems, such as tooth-rack with pinion, seem to be possible to operate the gates, but to operate these a drive system is also needed such as mentioned at the wire rope system.

For the wider gates (50 and 100 m) it looks less sensible to use a wire rope system or other mechanical operation system. The loads are very high, and it will result in many wire ropes/components which requires a lot of space and can fail.

A hydraulic operation system seems more suitable for the gate operation. The main advantage is the direct drive with the cylinders and the hydraulic unit with the oil to serve the cylinders can be in a suitable location not in the immediate area of the cylinders. When lowering the gate, a pulling force is exerted on the cylinders. By discharging the oil in a controlled manner from the cylinder using the pulling force, the gates can be lowered under their own weight. A great advantage of the hydraulic system, in case of failure of the controlled components, is the easy implantation of redundant manually operated valves to bypass failed components and to lower the gate manually. The synchronous moving of the gate must then be checked visually and controlled manually.

The number of components used when lowering the gate with a hydraulic system is less than when using a mechanical operation mechanism and the options to recover a failure are a lot easier.

A8.2 Alternative B: flap gate

The flap gate is introduced in the technical report of the sudden closure [1]. The idea is the self-closing mechanism. But the use of self-closing flap gate should be investigated further. It is highly likely that the uncontrolled motion of the gate will cause serious damage to the gate upon impact with the water or by clashing into the civil structure in close position. For now, it will be assumed that the gates need guidance from an operation system for a controlled closure.

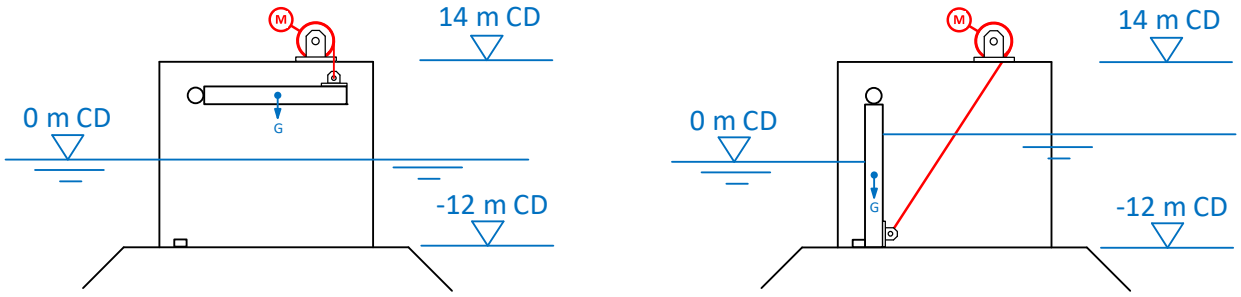


Figure 8.30 Flap gate (20 m high)

A8.2.1 Wire ropes

An easy-to-use operation system for the flap gate seems to be a wire system. When carried out as shown on Figure *8.30*, the maximum force in the wires is found in open position of the gate. When lowering the gate, the lever of the centre of mass of the gate and the wire rope will both decrease, where the lever of the centre of mass will decrease faster.

However, at a certain moment when closing the gates, the gates will reach the water level. When the gates are closed too slowly, hydraulic pressure will work on the gate increasing the loads on the wire rope.

When dimensioning the wire ropes in the open position of the gate, at water level differences of 0,2 m for the 20 m high gate and 0,35 m for the 15 m high gate, the load on the ropes because of the hydraulic pressure will exceed the load on open position of the gates. This is presented in the figure below, in which the wire loads are shown. In the table (F) the loads are presented, α_g is the closing angle, W the width, wh represents the submersion of the gate and ΔH the water level difference across the gate. The indices 20 and 15 represent the data for both the heights.

Please note that the loads on the wire ropes will increase rapidly with increasing water level differences.

$\Delta H_{20} = 0.2\ m \qquad W^T = [20.0\ \ 25.0\ \ 50.0\ \ 100.0]\ m$

$$\alpha_g = \begin{bmatrix} 0.0 \\ 15.0 \\ 30.0 \\ 45.0 \\ 60.0 \\ 75.0 \\ 90.0 \end{bmatrix} deg \qquad F_{wr20} = \begin{bmatrix} 1161 & 1451 & 2903 & 5805 \\ 1087 & 1359 & 2718 & 5436 \\ 1014 & 1268 & 2536 & 5072 \\ 1159 & 1449 & 2899 & 5797 \\ 1076 & 1344 & 2689 & 5378 \\ 850 & 1062 & 2125 & 4250 \\ 485 & 606 & 1212 & 2424 \end{bmatrix} kN \qquad wh_{20} = \begin{bmatrix} 0.0 \\ 0.0 \\ 0.0 \\ 4.9 \\ 8.5 \\ 10.9 \\ 12.0 \end{bmatrix} m$$

$\Delta H_{15} = 0.35\ m \qquad W^T = [20.0\ \ 25.0\ \ 50.0\ \ 100.0]\ m$

$$\alpha_g = \begin{bmatrix} 0.0 \\ 15.0 \\ 30.0 \\ 45.0 \\ 60.0 \\ 75.0 \\ 90.0 \end{bmatrix} deg \qquad F_{wr15} = \begin{bmatrix} 898 & 1122 & 2245 & 4489 \\ 867 & 1084 & 2168 & 4336 \\ 809 & 1011 & 2022 & 4044 \\ 839 & 1049 & 2098 & 4196 \\ 887 & 1109 & 2219 & 4437 \\ 767 & 959 & 1918 & 3836 \\ 495 & 619 & 1237 & 2475 \end{bmatrix} kN \qquad wh_{15} = \begin{bmatrix} 0.0 \\ 0.0 \\ 0.0 \\ 1.3 \\ 4.1 \\ 6.1 \\ 7.0 \end{bmatrix} m$$

Figure 8.31 Loads on the wire ropes

For holding the gates in open position, the following number of wire ropes with a diameter of 60 mm and a tensile strength of 1960 MPa is required as presented in Figure 8.32.

Assume a wire rope will be used

$k_{b.0} := 3.55$ Service factor wire ropes for ULS (K2 NEN3508)

$$F_{rope.0.20} := \frac{k_{b.0} \cdot m_{20} \cdot g}{2 \cdot 2} = \begin{bmatrix} 1740.7 \\ 2175.9 \\ 4351.7 \\ 8703.4 \end{bmatrix} kN \qquad F_{rope.0.15} := \frac{k_{b.0} \cdot m_{15} \cdot g}{2 \cdot 2} = \begin{bmatrix} 1305.5 \\ 1631.9 \\ 3263.8 \\ 6527.6 \end{bmatrix} kN$$

Assume steel wire rope with maximum diameter 60 mm and tensile strength 1960 MPa

$F_{br} := 2500 \cdot kN$ Minimum breaking force

Number of wire ropes per side of the gate

$$n_{rope.20} := ceil\left(\frac{F_{rope.0.20}}{F_{br}}\right) = \begin{bmatrix} 1.0 \\ 1.0 \\ 2.0 \\ 4.0 \end{bmatrix} \qquad n_{rope.15} := ceil\left(\frac{F_{rope.0.15}}{F_{br}}\right) = \begin{bmatrix} 1.0 \\ 1.0 \\ 2.0 \\ 3.0 \end{bmatrix}$$

Figure 8.32 Number of wire ropes

A8.3 Alternative C: radial gates

The radial gate was not mentioned in the sudden closure report [1]. But this can also be a feasible solution because of the independence of the operation forces with the hydraulic pressure on the gate. Therefore, the radial gate is considered as well. Pre-condition is that the gate should fit in the devised dimensions of the caissons.

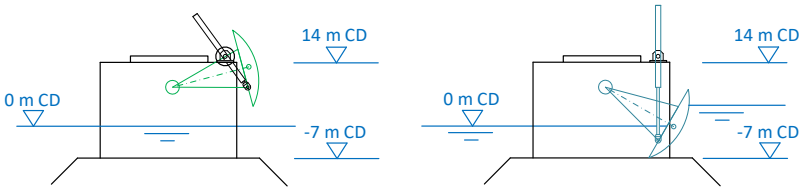


Figure 8.33 Radial gate (15 m high)

A 20 m gate will be difficult to fit, but a 15 m gate seems to be possible when the closed gate is positioned under a 30-degree angle. This will result in the main dimensions as shown in the figure below.

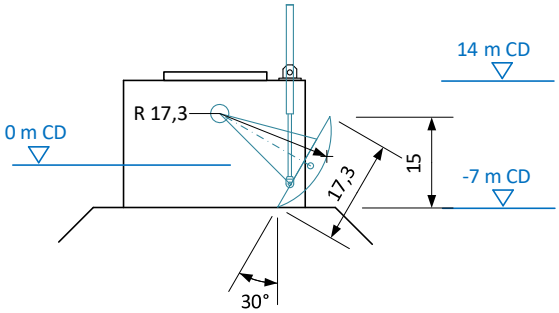


Figure 8.34 Main dimensions 15 m gate

Disadvantage is the location of the centre of mass of the gate. The operation loads are high, but independent of the hydraulic pressure on the gate. Meaning the gate can fully close under its own weight while at maximum hydraulic load.

Make note: in chapter 3, at the evaluation of the different gate-types, mentioned was the disadvantage of the pivot point under water level for the radial gate. As shown, it will be possible to have the pivot point above water level, making the radial gate actually score even better than the other gate-types.

A8.3.1 Operation system

The preferred operation mechanism seems to be an operation system using hydraulic cylinders (see section A8.1.5). Since there are only pulling loads on the hydraulic cylinders, there will be no buckling issue.

When using two cylinders per gate (one at each side) following standardized dimensions will be found as shown in the table below.

Table 8.30 *Standardised cylinders*

	Width of the gate							
	20 m		25 m		50 m		100 m	
	bore	rod	bore	rod	bore	rod	bore	rod
Height	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
15 m	500	360	550	380	800	560	1100	800

Maximum cylinder load

$\gamma_U := 1.5$ Loadfactor ULS

$$F_{cil} := \frac{1}{2} \cdot \gamma_U \cdot F_{h.max} = \begin{bmatrix} 1926.9 \\ 2408.6 \\ 4817.2 \\ 9634.3 \end{bmatrix} \text{kN}$$

Assume maximum cilinder pressure of 200 bar

$p_{cil} := 200 \cdot \text{bar}$ $\varphi := 2$

$$d_{z15} := \sqrt{\frac{4 \cdot F_{cil}}{\pi \cdot p_{cil} \cdot \left(1 - \frac{1}{\varphi}\right)}} = \begin{bmatrix} 495.3 \\ 553.8 \\ 783.2 \\ 1107.6 \end{bmatrix} \text{mm} \quad d_{s15} := \frac{1}{\sqrt{\varphi}} \cdot d_{z15} = \begin{bmatrix} 350.2 \\ 391.6 \\ 553.8 \\ 783.2 \end{bmatrix} \text{mm} \quad p := \frac{F_{cil}}{\frac{\pi}{4} \cdot \left(d_{z15}^2 - d_{s15}^2\right)} = \begin{bmatrix} 200.0 \\ 200.0 \\ 200.0 \\ 200.0 \end{bmatrix} \text{bar}$$

Figure 8.35 Theoretical cylinder dimensions

A8.4 Conclusions and recommendations

The flap gate is possible, but only a short time span is available for the closure of the gates. With the increasing water level difference, the loads on the closing gates and the operation mechanism will increase very quickly. Testing the gates will be very risky because of this short time span.

The vertical lift gate seems feasible. Up to a water level difference of about 3 m the gates will close under their own weight. After that point, the friction forces of the sliding strips will exceed the weight of the gate. The maximum pulling force at the operation mechanism is found in open position of the gate. Pushing the gate down, with for example hydraulic cylinders, at a higher water level difference will be difficult because of buckling problems. Testing the gates will be possible withing a water level difference of 3 m.

A radial gate seems feasible as well. The main advantage is its independence of the loads on the operation mechanism with respect to the hydraulic pressure on the gate. The disadvantage is the location of the centre of mass of the gate which results in high operation loads. Because of the independence with respect to the hydraulic pressure on the gates, the gates are easy to test, with a low power request. A 20 m high gate doesn't seem to fit in the dimensions of caissons.

The considered lift gate and radial gate are obvious types for frequently operated structures. In this dam, they are intended for single-use and therefore may be somewhat luxurious. It is therefore recommended that, in a further design phase, other solutions that are more appropriate for single-use closure should also be considered.

Possibilities of abandoning the gate operations mechanisms should also be considered. Now, it is assumed that due to the speed that the gate hits the water surface and later the bottom of the concrete closure element when only use gravity, it will suffer severe damage and therefore a motion work is necessary. But perhaps solutions without operation mechanism may be possible.

A9 RELIABILITY ANALYSIS

For the sudden closure of the dam, the probability of failure to close is important to know. For the study some technical details are necessary. In this chapter the electro- and mechanical components of the gate are considered in more detail, to be able to make an estimate of the probability of failure.

A9.1 Operation mechanisms

The vertical lift gate, the flap gate and the radial gates will be considered in this phase, the catamaran appears too unreliable without further investigation. For the vertical lift-gate and the radial gate a hydraulic operation mechanism with cylinders will be considered, for the flap gate a wire rope system.

A9.1.1 Hydraulic operation mechanism

In Appendix A11, a principal scheme of a hydraulic operation mechanism is shown. Every gate contains two cylinders to operate the gate. The cylinders are equipped with a position detection system which will be used for synchronous operation of the cylinders. When one of the cylinders stays behind the velocity of the forward running cylinder will be decreased to synchronise the cylinders. Besides the position detection system, the cylinders will be provided with limit switches as backup system to detect the retarder and end positions of the gate when the position detection system will fail. In the figure below the principal of the cylinder configuration connected to the gates are shown.

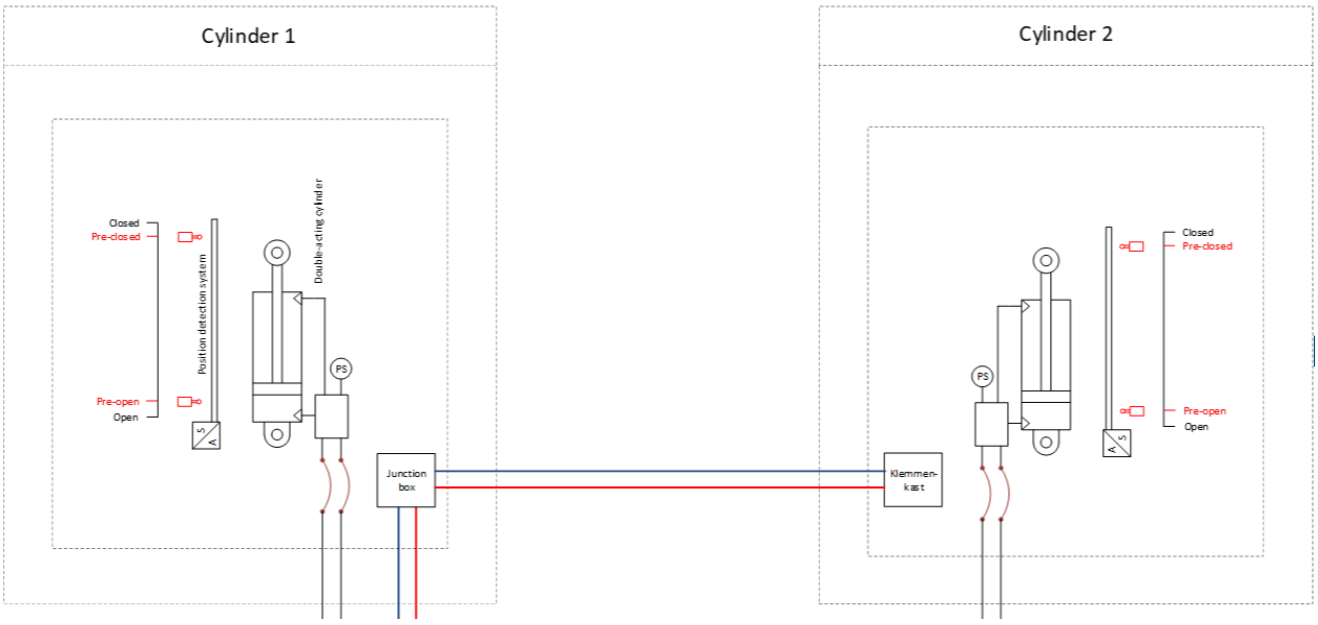


Figure 9.36 Cylinders of the gate

The cylinders are equipped with a valve-block with safety and direction valves. An example of the valve-block is given in the figure below. In case of an emergency the safety and differential valves can be energized, and the gate will close uncontrolled.

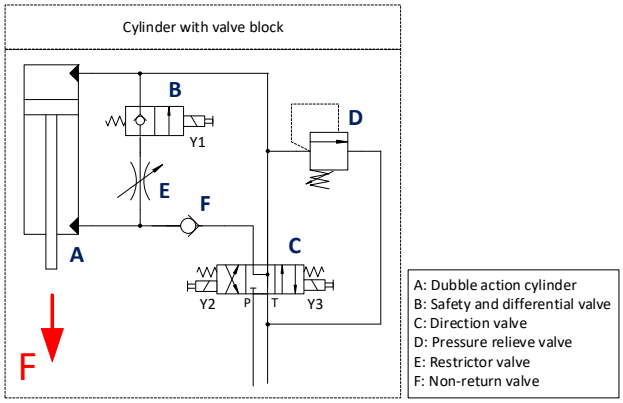


Figure 9.37 Valve block on the cylinder

With a hydraulic Power Unit (HPU) the gates can be closed controlled. The HPU will also be necessary for testing purposed. The gates should be tested regularly for a reliable closure. For the estimation of the probability of failure, a controlled closure with the HPU will be assumed under normal conditions. Closure with the safety valve (B) under the weight of the gate will be a recovery action. The described operation mechanism will be assumed for all sizes of the gates (20, 25, 50 and 100 m). The principle stays the same, the dimensions of the system will differ.

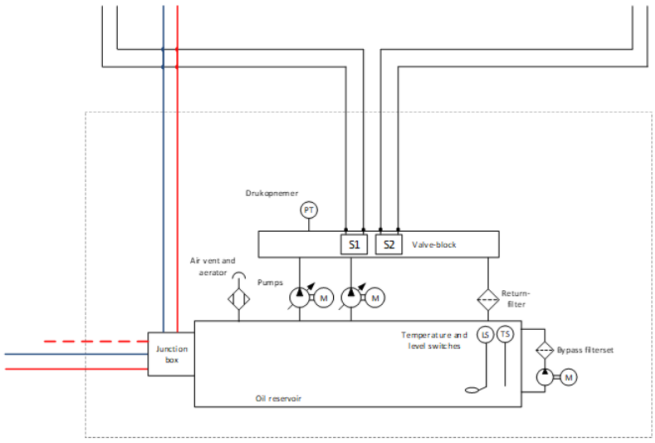


Figure 9.38 Hydraulic power unit

A9.1.2 Electromechanical operation mechanism

In Annex C principles of an electromechanical system is given. Different from hydraulic operation system, the configuration differs per size of the gate. The figure below shows the assumptions for the estimation of the probability of failure, a controlled closure with the HPU will be assumed under normal conditions. Closure with the safety valve (B) under the weight of the gate will be a recovery action. The described operation mechanism will be assumed for all sizes of the gates (20, 25, 50 and 100 m). The principle stays the same, the dimensions of the system will differ.

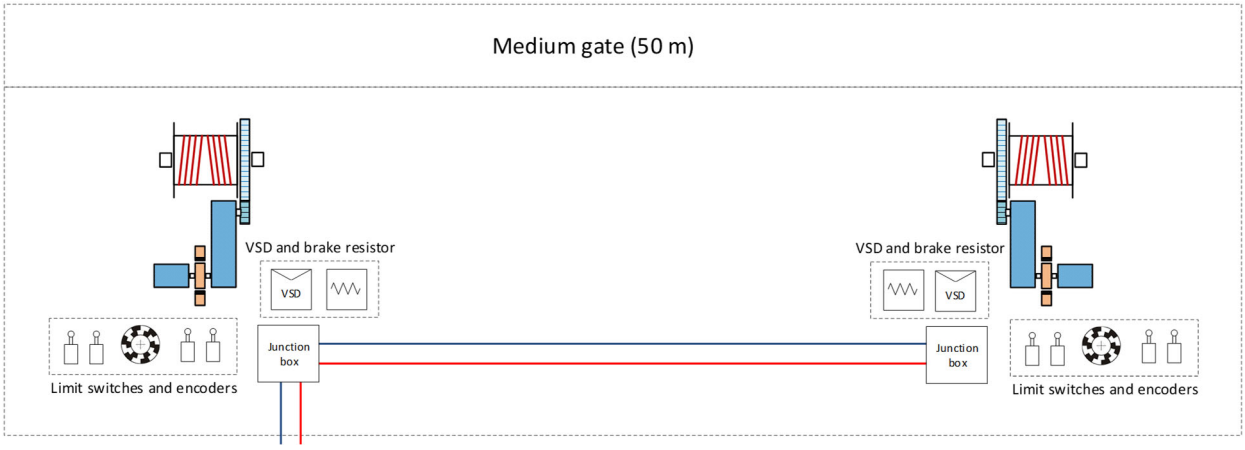


Figure 9.39 Operation mechanism 50 m flap gate

On all gate widths (20, 25, 50 and 100 m), it is assumed that the wire ropes are connected to both sides of the gates. The number of wire ropes varies with the width of the gate.

A9.2 Energy supply

For the operation mechanism electoral power will be necessary although the gates are basically self-closing due to the overweight. Therefore, the power request will be relatively low. Because of the length of the 30 km dam, it will not be possible to serve the dam completely with low voltage on which the operation mechanism will run. Therefore, transformers on the dam will be used to transform the medium voltage incoming power to low voltage. It will be assumed; each transformer can serve a length of 1500 m caissons. An emergency power generator set will be assumed to provide electric power in case of failure of the power connection.

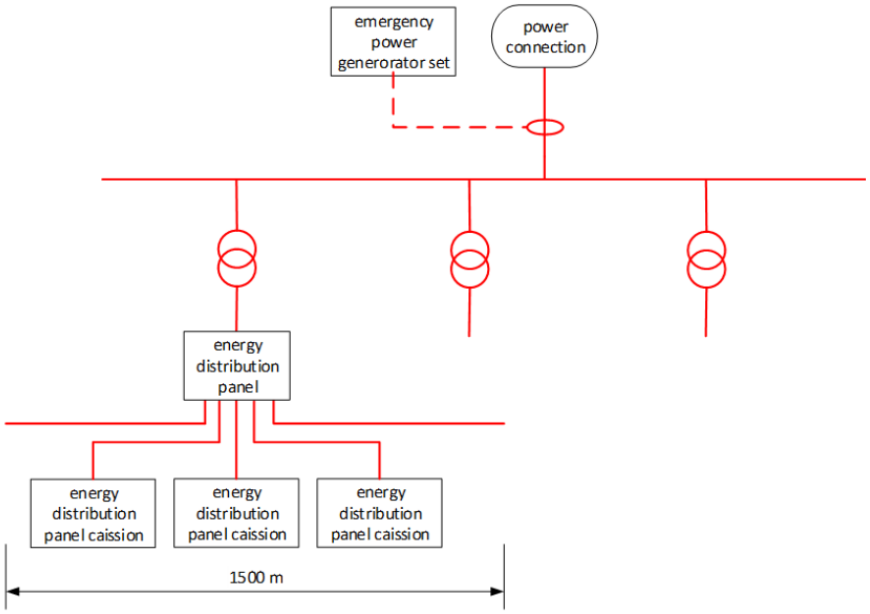


Figure 9.40 Main electrical distribution

A9.3 Local gate energy distribution and control and operating

Assumed is an energy distribution panel and control system per caisson. The figure below shows the principle with shown only one connection to the operation mechanism to a gate, in this case gate A. For each concrete closure element an energy distribution panel, control system and UPS-system will be assumed to serve all the gates in the caisson. The UPS-system will be necessary to keep the control system in operation in cased of power failure and the switchover to emergency power. A local control panel will be used to operate the gates in the caisson.

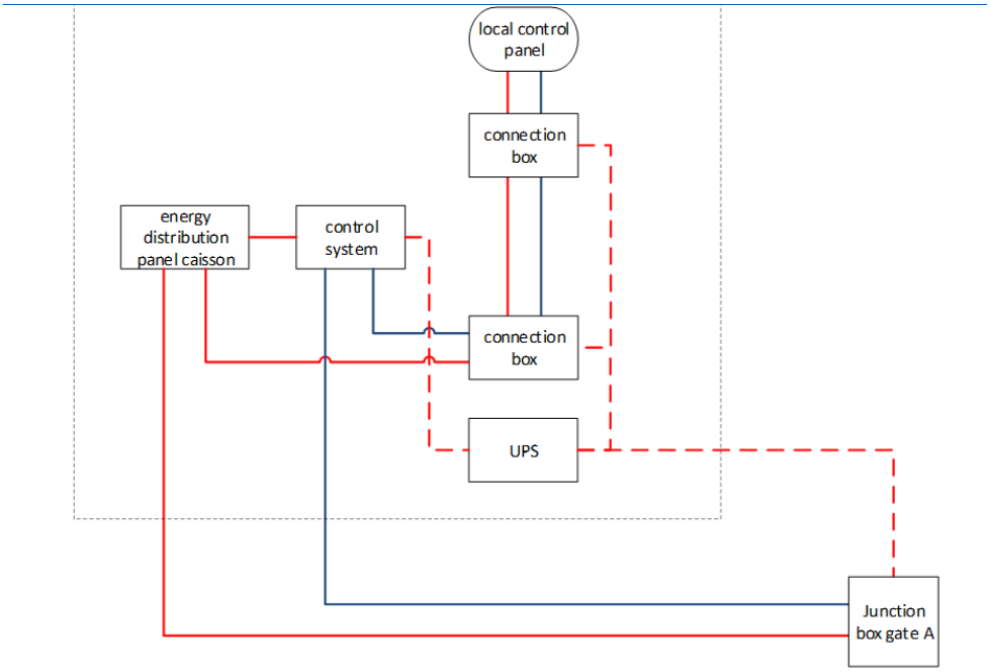


Figure 9.41 Local gate energy distribution and control and operating

A9.4 Reliability estimate

Assumptions:

- The month before the sudden closure all the gates will be tested on proper functioning.
- The hydraulic cylinders are equipped with an emergency valve to close the gate in case of malfunction of the system manually. For this emergency closure with the emergency valve an equivalent function recovery time of 2 hours is assumed.

Failure of the dam
The dam consists of a combination of linked caissons. The caissons contain gates that all need to be closed at the same time. The different types of gates are explained in in the paragraphs before.
The gates close under their own weight but to prevent the risk of clamping of the gates in the structure, the closing needs to be controlled. In the following paragraphs the different types of closing mechanisms are explained and the way they can fail.

Hydraulic closing mechanism.
The controlled closing of the gates can be done using hydraulic cylinders. Every gate will be equipped with two cylinders, one on each site of the gate. When the gates are open the hydraulic cylinders are retracted. When the command for the sudden closure is given, valves are opened, and the gate will close under its

own weight. By controlling the flow of hydraulic oil that leaves the cylinders, the closing process can be adjusted if necessary. To control the flow in both cylinders sensors and a control system is needed.

Assumptions hydraulic closing mechanism:
- The gates in the concrete closure element will be equipped with two cylinders, one on each side of the gate.
- Every concrete closure element will be equipped with its own control system.
- A hydraulic power unit is not needed for the closing of the gates when testing on proper operation.

Electromechanical mechanism
The controlled closing of the gates can also be done using an electromechanical winch system. Every gate will be equipped whit a winch system on each side of the gate. When the command for the sudden closure is given, the gate will close under its own weight. By adjusting the speed on the winches on both sides of the gate, the closure will take place without clamping.

Assumptions electromechanical mechanism
- A concrete closure element can contain one, two or four gates.
- The gate will be equipped with a winch system on each side of the gate.
- Every concrete closure element will be equipped with its own control system.

Assumptions
- Building of the dam (approximately 30 kilometers) will take years. While the components of the last caissons will be almost brand new, the components of the first caissons will be in a salty environment for years. It is important that these components will be protected from the elements so we can assume that they will still perform as if they are almost new.
- As mentioned before, the building of the dam will take years. When the construction of the dam there will be a moment when the sudden closure will take place. It is important that al the gates and machinery will be tested thoroughly so the change of failures during the sudden closure is minimized.

For the reliability analysis different failure data sets are used. In the model, to determine the quantitative results, often generic available failure data is applied. The following list shows the different data sets, the list is in random order and not arranged by priority in application from top to bottom:
- Specific failure data from suppliers.
- Generic failure data from ‘Leidraad kunstwerken’ (Guideline structures), TAW Technical Advisory Committee for Flood Defences, 2003;
- Generic failure data from ‘OREDA’, Det Norske Veritas (DNV), 2002/2009;
- Generic failure data from ‘Methods for determining and processing probabilities’, The Committee for the prevention of disasters by hazardous materials, 1997;
- Expert Judgment.

Software
The following software is used for this analysis:
- Reliability Workbench (RWB) v15

Failure analysis
The closing systems are modelled in RWB using the assumptions mentioned before and according to the previous descriptions.

The results of the failure analysis are showed in the figure below.

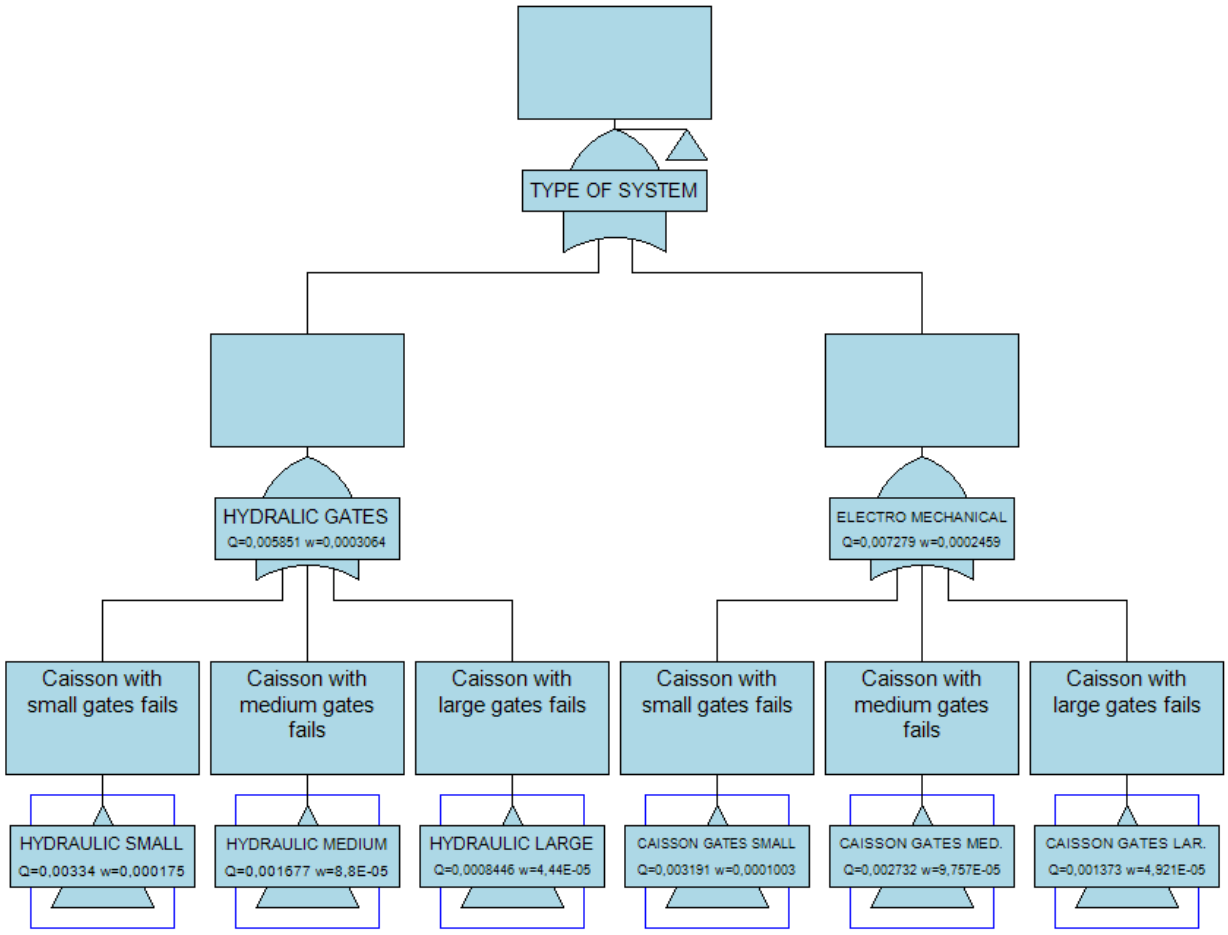


Figure 9.42 Results failure analysis

The complete fault tree analysis can be found in annex D.

Results

In the table below, the results main results are given from the fault tree analysis. These results will be used for the probability calculations.

Table 8.31 Results failure analysis

	Hydraulic closing system		Electromechanical closing system	
Gate size:	Unavailability (Q)	Failure frequency (W) (1/ h)	Unavailability (Q)	Failure frequency (W) (1/ h)
Small	3.34E-03	1.75E-04	3.19E-03	1.00E-04
Medium	1.68E-03	8.79E-05	2.73E-03	9.76E-05
Large	8.45E-04	4.44E-05	1.37E-03	4.92E-05

The results from the analyses, unavailability (Q) and failure frequency (λ) will be used in the following formula:

$$U = \frac{1}{2} \cdot \lambda_s \cdot T + \alpha \cdot \lambda_b \cdot \theta$$

- U = Probability of failure
- λ_s = Failure frequency standby (0.1 x λ_b) (it is assumed that the failure frequency in standby modus is an order smaller than in operation)
- T = Test interval
- α = Operation (fraction per year)
- λ_b = Failure frequency during operation
- θ = equivalent repair time (unavailability / failure frequency)

With this furmula the probability of failure can be calculated for the differt gate sizes and different closing mechanisms, with:

- T = 730 hours (test period). It is assumed that one month before the sudden closure all the gates will be tested on proper functioning.
- α = 6 / 8760 (assumed fraction per year the operation mechanism is in operation)

For the hydraulic closing mechanism this will result in:

Table 8.32 Probability of failure hydraulic gates

Gate size:	Probability of failure
Small (25 m)	0.0064
Medium (50 m)	0.0032
Large (100 m)	0.0016

The value for the probability of failure of a small gate will be used to calculate the probability that a dam of 10, 15 or 20 km will fail. For example: The probability that one small gate will fail is: 1 / 0.0064 = 156.25. So the chance that one out of four gates in one concrete closure element will fail is: 1 / (156.25 · 4) = 0.0016

Table 8.33 Probabilities for different dam lengths - hydraulic operated gates

gate configuration		25 m gates			50 m gates		
length	number of closure elements	number of gates	probability of failure	number of gates failing in closure	number of gates in ca	probability of failure	number of gates failing in closure
100 m	1	4	0.0016	1	2	0.002	1
10 km	100	400	0.641	1	200	0.321	1
15 km	150	600	0.962	1	300	0.481	1
20 km	200	800	1.282	2	400	0.641	1

For the electromechanical closing mechanism this will result in:

This will lead to the following results:

Table 8.34 Probability of failure electromechanical

Gate size:	Probability of failure
Small (25 m)	0.0073
Medium (50 m)	0.0071
Large (100 m)	0.0036

The value for the probability of failure of a small gate will be used to calculate the probability that a dam of 10, 15 or 20 km will fail. For example:

The probability that one small gate will fail is: 1 / 0.0073 = 137. So the chance that one out of four gates in one concrete closure element will fail is: 1 / (137 · 4) = 0.0018

Table 8.35 Probabilities for different dam lengths - electromechanical operated gates

gate configuration		25 m gates			50 m gates		
length	number of closure elements	number of gates	probability of failure	number of gates failing in closure	number of gates in ca	probability of failure	number of gates failing in closure
100 m	1	4	0.0018	1	2	0.0018	1
10 km	100	400	0.7326	1	200	0.366	1
15 km	150	600	1.0989	2	300	0.549	1
20 km	200	800	1.4653	2	400	0.733	1

Conclusion
While taking the assumptions into account, the following can be concluded:

- Only one hydraulic operated gate of 25 m will fail in dam with length a of: 10 and 15 km.
- Two hydraulic operated gates of 25 m will fail in dam with length a of: 20 km
- Only one hydraulic operated gate of 50 m will fail in dam with length a of: 10, 15 and 20 km.
- Only one electromechanical operated gate of 25 m will fail in a dam with a length of: 10 km.
- Two electromechanical operated gates of 25 m will fail in a dam with a length of: 15 and 15 km.
- Only one electromechanical operated gate of 50 m will fail in a dam with a length of: 20 km.

A10 INDICATIVE CALCULATIONS

Royal HaskoningDHV

KALPASAR RISK

Mechanical

Project: Kalpasar
Author: GJWH van der Sanden
Version: 01
Date: 09.01.2023

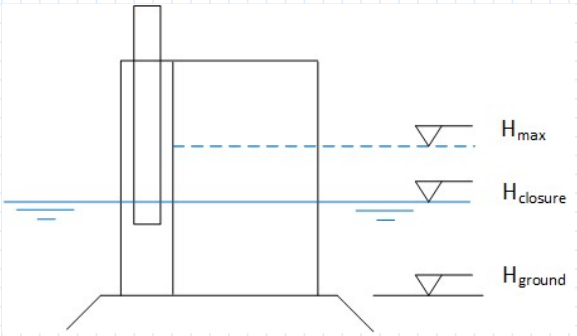
Legend

- Parameter to be filled in
- Result
- Unity Check
- To Check
- Constant

References:

[1] NEN 6786:2001 - VOBB

Data



$H_{max} := 6 \cdot m$

$H_{closure} := 0 \cdot m$

$H_{gr20} := -12 \cdot m$

$H_{gr15} := -7 \cdot m$

$H_{20} := 20 \cdot m$

$H_{15} := 15 \cdot m$

$\rho_w := 1030 \cdot \frac{kg}{m^3}$

$m_g := 500 \cdot \frac{kg}{m^2}$

$W := \begin{bmatrix} 20 \\ 25 \\ 50 \\ 100 \end{bmatrix} \cdot m$

$m_{20} := H_{20} \cdot W \cdot m_g = \begin{bmatrix} 200000.0 \\ 250000.0 \\ 500000.0 \\ 1000000.0 \end{bmatrix}$ kg

$m_{15} := H_{15} \cdot W \cdot m_g = \begin{bmatrix} 150000.0 \\ 187500.0 \\ 375000.0 \\ 750000.0 \end{bmatrix}$ kg

$n := 10$ $\quad i := 1..n+1$ $\quad dH_i := \frac{H_{max} - H_{closure}}{n} \cdot (i-1)$ $\quad p_i := dH_i \cdot \rho_w \cdot g$

$\mu := 0.2$ $\quad z := 1..\text{length}(W)$

$F_{w20_{i,z}} := \mu \cdot p_i \cdot W_z \cdot (dH_i - H_{gr20})$ $\quad F_{w15_{i,z}} := \mu \cdot p_i \cdot W_z \cdot (dH_i - H_{gr15})$

$F_{w20} = \begin{bmatrix} 0.0 & 0.0 & 0.0 & 0.0 \\ 305.4 & 381.8 & 763.6 & 1527.2 \\ 640.0 & 800.0 & 1600.0 & 3199.9 \\ 1003.6 & 1254.5 & 2509.1 & 5018.1 \\ 1396.3 & 1745.4 & 3490.9 & 6981.7 \\ 1818.2 & 2272.7 & 4545.4 & 9090.8 \\ 2269.1 & 2836.3 & 5672.6 & 11345.3 \\ 2749.0 & 3436.3 & 6872.6 & 13745.2 \\ 3258.1 & 4072.7 & 8145.3 & 16290.7 \\ 3796.3 & 4745.4 & 9490.8 & 18981.5 \\ 4363.6 & 5454.5 & 10908.9 & 21817.8 \end{bmatrix}$ kN

$F_{w15} = \begin{bmatrix} 0.0 & 0.0 & 0.0 & 0.0 \\ 184.2 & 230.3 & 460.6 & 921.2 \\ 397.6 & 497.0 & 993.9 & 1987.8 \\ 640.0 & 800.0 & 1600.0 & 3199.9 \\ 911.5 & 1139.4 & 2278.8 & 4557.5 \\ 1212.1 & 1515.1 & 3030.3 & 6060.5 \\ 1541.8 & 1927.2 & 3854.5 & 7709.0 \\ 1900.6 & 2375.7 & 4751.4 & 9502.9 \\ 2288.4 & 2860.6 & 5721.1 & 11442.2 \\ 2705.4 & 3381.8 & 6763.5 & 13527.1 \\ 3151.5 & 3939.3 & 7878.7 & 15757.3 \end{bmatrix}$ kN

$R_{20}^{\langle z \rangle} := g \cdot m_{20_z} - F_{w20}^{\langle z \rangle}$ $\quad R_{15}^{\langle z \rangle} := g \cdot m_{15_z} - F_{w15}^{\langle z \rangle}$

$R_{20} = \begin{bmatrix} 1961.3 & 2451.7 & 4903.3 & 9806.7 \\ 1655.9 & 2069.9 & 4139.7 & 8279.4 \\ 1321.3 & 1651.7 & 3303.4 & 6606.7 \\ 957.7 & 1197.1 & 2394.3 & 4788.5 \\ 565.0 & 706.2 & 1412.5 & 2824.9 \\ 143.2 & 179.0 & 357.9 & 715.9 \\ -307.7 & -384.7 & -769.3 & -1538.6 \\ -787.7 & -984.6 & -1969.3 & -3938.6 \\ -1296.8 & -1621.0 & -3242.0 & -6484.0 \\ -1835.0 & -2293.7 & -4587.4 & -9174.9 \\ -2402.2 & -3002.8 & -6005.6 & -12011.2 \end{bmatrix}$ kN

$R_{15} = \begin{bmatrix} 1471.0 & 1838.7 & 3677.5 & 7355.0 \\ 1286.8 & 1608.4 & 3216.9 & 6433.8 \\ 1073.4 & 1341.8 & 2683.6 & 5367.1 \\ 831.0 & 1038.8 & 2077.5 & 4155.0 \\ 559.5 & 699.4 & 1398.7 & 2797.5 \\ 258.9 & 323.6 & 647.2 & 1294.5 \\ -70.8 & -88.5 & -177.0 & -354.0 \\ -429.6 & -537.0 & -1073.9 & -2147.9 \\ -817.5 & -1021.8 & -2043.6 & -4087.3 \\ -1234.4 & -1543.0 & -3086.0 & -6172.1 \\ -1680.5 & -2100.6 & -4201.2 & -8402.3 \end{bmatrix}$ kN

$$dH = \begin{bmatrix} 0.0 \\ 0.6 \\ 1.2 \\ 1.8 \\ 2.4 \\ 3.0 \\ 3.6 \\ 4.2 \\ 4.8 \\ 5.4 \\ 6.0 \end{bmatrix} \text{ m}$$

conclusion: up to a water level difference of 3 m, the gates can close at their own weight

Maximum pressing force

$P_{20_z} := \max\left(R_{20}^{(z)}\right) = \begin{bmatrix} 1961.3 \\ 2451.7 \\ 4903.3 \\ 9806.7 \end{bmatrix}$ kN $\quad P_{15_z} := \max\left(R_{15}^{(z)}\right) = \begin{bmatrix} 1471.0 \\ 1838.7 \\ 3677.5 \\ 7355.0 \end{bmatrix}$ kN $\quad W = \begin{bmatrix} 20.0 \\ 25.0 \\ 50.0 \\ 100.0 \end{bmatrix}$ m

Cylinder force, assuming 2 cylinders per gate

$F_{cil20} := \frac{P_{20}}{2} = \begin{bmatrix} 980.7 \\ 1225.8 \\ 2451.7 \\ 4903.3 \end{bmatrix}$ kN $\quad F_{cil15} := \frac{P_{15}}{2} = \begin{bmatrix} 735.5 \\ 919.4 \\ 1838.7 \\ 3677.5 \end{bmatrix}$ kN

Operation forces

It is assumed that the slide has movement on two sides.

$F_{Ed.O.max.20} := \frac{1}{2} \cdot \left(\overset{\frown}{R_{20}^{1}}\right)^T = \begin{bmatrix} 980.7 \\ 1225.8 \\ 2451.7 \\ 4903.3 \end{bmatrix}$ kN $\quad F_{Ed.O.max.15} := \frac{1}{2} \cdot \left(\overset{\frown}{R_{15}^{1}}\right)^T = \begin{bmatrix} 735.5 \\ 919.4 \\ 1838.7 \\ 3677.5 \end{bmatrix}$ kN

Assume maximum cilinder pressure of 200 bar

$p_{cil} := 200 \cdot bar$ $\quad \varphi := 2$ $\quad \gamma_U := 1.5$ Loadfactor ULS

Piston diameter

$$d_{z20} := \sqrt{\frac{4 \cdot \gamma_U \cdot F_{Ed.O.max.20}}{\pi \cdot p_{cil} \cdot \left(1 - \frac{1}{\varphi}\right)}} = \begin{bmatrix} 432.8 \\ 483.9 \\ 684.3 \\ 967.7 \end{bmatrix} \text{ mm}$$

Rod diameter

$$d_{s20} := \frac{1}{\sqrt{2}} \cdot d_{z20} = \begin{bmatrix} 306.0 \\ 342.1 \\ 483.9 \\ 684.3 \end{bmatrix} \text{ mm}$$

Pressure

$$p := \frac{\gamma_U \cdot F_{Ed.O.max.20}}{\frac{\pi}{4} \cdot \left(d_{z20}^{\,2} - d_{s20}^{\,2}\right)} = \begin{bmatrix} 200.0 \\ 200.0 \\ 200.0 \\ 200.0 \end{bmatrix} \text{ bar}$$

$$d_{z15} := \sqrt{\frac{4 \cdot \gamma_U \cdot F_{Ed.O.max.15}}{\pi \cdot p_{cil} \cdot \left(1 - \frac{1}{\varphi}\right)}} = \begin{bmatrix} 374.8 \\ 419.0 \\ 592.6 \\ 838.1 \end{bmatrix} \text{ mm} \quad d_{s15} := \frac{1}{\sqrt{2}} \cdot d_{z15} = \begin{bmatrix} 265.0 \\ 296.3 \\ 419.0 \\ 592.6 \end{bmatrix} \text{ mm} \quad p := \frac{\gamma_U \cdot F_{Ed.O.max.15}}{\frac{\pi}{4} \cdot \left(d_{z15}^{\,2} - d_{s15}^{\,2}\right)} = \begin{bmatrix} 200.0 \\ 200.0 \\ 200.0 \\ 200.0 \end{bmatrix} \text{ bar}$$

Estimation of the buckling force

$E_s := 210 \cdot 10^3 \cdot MPa$ $\quad I_{s20} := \frac{\pi}{64} \cdot d_{s20}^{\,4}$ $\quad I_{s15} := \frac{\pi}{64} \cdot d_{s15}^{\,4}$

$l_{buc20} := 2 \cdot H_{20} + 2 \cdot m$ $\quad l_{buc15} := 2 \cdot H_{15} + 2 \cdot m$ $\quad sf := 2$ Safety factor

$$F_{E20} := \frac{\pi^2 \cdot E_s}{sf} \cdot \frac{I_{s20}}{l_{buc20}^2} = \begin{bmatrix} 252.9 \\ 395.2 \\ 1580.6 \\ 6322.4 \end{bmatrix} kN \qquad F_{E15} := \frac{\pi^2 \cdot E_s}{sf} \cdot \frac{I_{s15}}{l_{buc15}^2} = \begin{bmatrix} 245.1 \\ 382.9 \\ 1531.6 \\ 6126.4 \end{bmatrix} kN$$

$$K_{20_{i,z}} := \frac{\gamma_U \cdot R_{20_{i,z}}}{F_{E20_z}} \qquad K_{15_{i,z}} := \frac{\gamma_U \cdot R_{15_{i,z}}}{F_{E15_z}}$$

$$W^T = [20.0 \ \ 25.0 \ \ 50.0 \ \ 100.0] \, m \qquad W^T = [20.0 \ \ 25.0 \ \ 50.0 \ \ 100.0] \, m$$

$$K_{20} = \begin{bmatrix} 11.63 & 9.31 & 4.65 & 2.33 \\ 9.82 & 7.86 & 3.93 & 1.96 \\ 7.84 & 6.27 & 3.13 & 1.57 \\ 5.68 & 4.54 & 2.27 & 1.14 \\ 3.35 & 2.68 & 1.34 & 0.67 \\ 0.85 & 0.68 & 0.34 & 0.17 \\ -1.83 & -1.46 & -0.73 & -0.37 \\ -4.67 & -3.74 & -1.87 & -0.93 \\ -7.69 & -6.15 & -3.08 & -1.54 \\ -10.88 & -8.71 & -4.35 & -2.18 \\ -14.25 & -11.40 & -5.70 & -2.85 \end{bmatrix} \quad K_{15} = \begin{bmatrix} 9.00 & 7.20 & 3.60 & 1.80 \\ 7.88 & 6.30 & 3.15 & 1.58 \\ 6.57 & 5.26 & 2.63 & 1.31 \\ 5.09 & 4.07 & 2.03 & 1.02 \\ 3.42 & 2.74 & 1.37 & 0.68 \\ 1.58 & 1.27 & 0.63 & 0.32 \\ -0.43 & -0.35 & -0.17 & -0.09 \\ -2.63 & -2.10 & -1.05 & -0.53 \\ -5.00 & -4.00 & -2.00 & -1.00 \\ -7.56 & -6.04 & -3.02 & -1.51 \\ -10.29 & -8.23 & -4.11 & -2.06 \end{bmatrix} \quad dH = \begin{bmatrix} 0.0 \\ 0.6 \\ 1.2 \\ 1.8 \\ 2.4 \\ 3.0 \\ 3.6 \\ 4.2 \\ 4.8 \\ 5.4 \\ 6.0 \end{bmatrix} m$$

Assume a wire rope will be used

$k_{b.O} := 3.55$ Service factor wire ropes for ULS (K2 NEN3508)

$$F_{rope.O.20} := k_{b.O} \cdot F_{Ed.O.max.20} = \begin{bmatrix} 3481.4 \\ 4351.7 \\ 8703.4 \\ 17406.8 \end{bmatrix} kN \qquad F_{rope.O.15} := k_{b.O} \cdot F_{Ed.O.max.15} = \begin{bmatrix} 2611.0 \\ 3263.8 \\ 6527.6 \\ 13055.1 \end{bmatrix} kN$$

Assume steel wire rope with maximum diameter 60 mm and tensile strength 1960 MPa

$F_{br} := 2500 \cdot kN$ Minimum breaking force

Number of wire ropes per side of the gate

$$n_{rope.20} := ceil\left(\frac{F_{rope.O.20}}{F_{br}}\right) = \begin{bmatrix} 2.0 \\ 2.0 \\ 4.0 \\ 7.0 \end{bmatrix} \qquad n_{rope.15} := ceil\left(\frac{F_{rope.O.15}}{F_{br}}\right) = \begin{bmatrix} 2.0 \\ 2.0 \\ 3.0 \\ 6.0 \end{bmatrix}$$

Cable drum

$d_w := 60 \cdot mm$ $h_1 := 18$

$D_d := d_w \cdot h_1 = 1080.0 \, mm$ Take diameter 1250 mm

Braking power

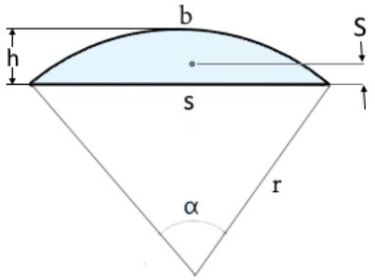
Assume closing time 30 minutes

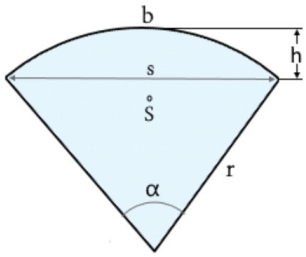
$t := 30 \cdot min$

$$v_{20} := \frac{H_{20}}{t} = 11.1 \, \frac{mm}{s} \qquad v_{15} := \frac{H_{15}}{t} = 8.3 \, \frac{mm}{s}$$

$$P_{20} := v_{20} \cdot \left(\widehat{R_{20}}\right)^T = \begin{bmatrix} 21.8 \\ 27.2 \\ 54.5 \\ 109.0 \end{bmatrix} \text{kW} \qquad P_{15} := v_{15} \cdot \left(\widehat{R_{15}}\right)^T = \begin{bmatrix} 12.3 \\ 15.3 \\ 30.6 \\ 61.3 \end{bmatrix} \text{kW} \qquad W = \begin{bmatrix} 20.0 \\ 25.0 \\ 50.0 \\ 100.0 \end{bmatrix} \text{m}$$

Radial gate





Formulas for calculating segments of a circle

Angle	$\alpha = 2 \cdot acos\left(1 - \frac{h}{r}\right)$
Height	$h = r \cdot \left(1 - cos\left(\frac{\alpha}{2}\right)\right)$
Area	$A = \frac{r^2}{2} \cdot \left(\frac{\alpha \cdot \pi}{180^\circ} - sin(\alpha)\right)$
Perimeter	$P = s + b$
Arc length	$b = r \cdot \pi \cdot \frac{\alpha}{180^\circ}$
Chord	$s = 2 \cdot r \cdot sin\left(\frac{\alpha}{2}\right)$
Centroid	$S = \frac{s^3}{12 \cdot A} - r \cdot cos\left(\frac{a}{2}\right)$

Formulas for calculating sectors of a circle

Angle	$\alpha = \frac{360 \cdot A}{r^2 \cdot \pi}$
	$\alpha = \frac{180 \cdot (P - 2 \cdot r)}{r \cdot \pi}$
	$\alpha = \frac{180 \cdot b}{r \cdot \pi}$
	$\alpha = 2 \cdot asin\left(\frac{s}{2 \cdot r}\right)$
Area	$A = r^2 \cdot \pi \cdot \frac{\alpha}{360^\circ}$
Arc length	$b = r \cdot \pi \cdot \frac{\alpha}{180^\circ}$
Perimeter	$P = 2 \cdot r + b$
Chord	$s = 2 \cdot r \cdot sin\left(\frac{\alpha}{2}\right)$
Centroid	$S = \frac{2}{3} \cdot \frac{r \cdot s}{b}$

$\alpha_g := 90 \cdot \text{deg}$

$s_g := 20 \cdot \text{m}$

$$r_g := \frac{s_g}{2 \cdot \sin\left(\frac{\alpha_g}{2}\right)} = 14.1 \text{ m}$$

$r_{gs} := 15 \cdot \text{m}$

$$s_{gs} := 2 \cdot r_{gs} \cdot \sin\left(\frac{\alpha_g}{2}\right) = 21.2 \text{ m}$$

$$\alpha_{gs} := 2 \cdot \text{asin}\left(\frac{s_g}{2 \cdot r_{gs}}\right) = 83.6 \text{ deg}$$

$$W = \begin{bmatrix} 20.0 \\ 25.0 \\ 50.0 \\ 100.0 \end{bmatrix} \text{m} \qquad m_{20} = \begin{bmatrix} 200000.0 \\ 250000.0 \\ 500000.0 \\ 1000000.0 \end{bmatrix} \text{kg} \qquad m_{15} = \begin{bmatrix} 150000.0 \\ 187500.0 \\ 375000.0 \\ 750000.0 \end{bmatrix} \text{kg}$$

$$\alpha_g := \begin{bmatrix} 15 \\ 30 \\ 45 \\ 60 \\ 75 \\ 90 \end{bmatrix} \cdot deg \qquad x_{G20} := \begin{bmatrix} 3.88 \\ 7.5 \\ 10.61 \\ 12.99 \\ 14.49 \\ 15 \end{bmatrix} \cdot m \qquad la_{20} := \begin{bmatrix} 12.68 \\ 14.76 \\ 14.93 \\ 14.26 \\ 13.11 \\ 11.61 \end{bmatrix} \cdot m$$

$q := 1 .. rows(x_{G20})$

$$F_{k20_{q,z}} := \frac{m_{20_z} \cdot g \cdot x_{G20_q}}{la_{20_q}} \qquad F_{k20} = \begin{bmatrix} 600.2 & 750.2 & 1500.4 & 3000.8 \\ 996.6 & 1245.8 & 2491.5 & 4983.1 \\ 1393.8 & 1742.3 & 3484.5 & 6969.1 \\ 1786.7 & 2233.3 & 4466.6 & 8933.3 \\ 2167.8 & 2709.7 & 5419.5 & 10838.9 \\ 2534.0 & 3167.5 & 6335.0 & 12670.1 \end{bmatrix} kN$$

$\alpha_g := 90 \cdot deg$

$s_g := 15 \cdot m$

$$r_g := \frac{s_g}{2 \cdot \sin\left(\frac{\alpha_g}{2}\right)} = 10.6\ m$$

$r_{gs} := 10.6 \cdot m$

$$s_{gs} := 2 \cdot r_{gs} \cdot \sin\left(\frac{\alpha_g}{2}\right) = 15.0\ m$$

$$\alpha_{gs} := 2 \cdot asin\left(\frac{s_g}{2 \cdot r_{gs}}\right) = 90.1\ deg$$

$$x_{G15} := \begin{bmatrix} 2.74 \\ 5.3 \\ 7.5 \\ 9.18 \\ 10.24 \\ 10.6 \end{bmatrix} \cdot m \qquad la_{15} := \begin{bmatrix} 8.83 \\ 10.36 \\ 10.64 \\ 10.23 \\ 9.40 \\ 8.27 \end{bmatrix} \cdot m$$

$$F_{k15_{q,z}} := \frac{m_{15_z} \cdot g \cdot x_{G15_q}}{la_{15_q}} \qquad F_{k15} = \begin{bmatrix} 456.5 & 570.6 & 1141.1 & 2282.3 \\ 752.5 & 940.7 & 1881.3 & 3762.7 \\ 1036.9 & 1296.1 & 2592.2 & 5184.4 \\ 1320.0 & 1650.0 & 3300.0 & 6600.1 \\ 1602.4 & 2003.1 & 4006.1 & 8012.2 \\ 1885.4 & 2356.8 & 4713.6 & 9427.2 \end{bmatrix} kN$$

Assume a wire rope will be used

$k_{b.O} := 3.55$ Service factor wire ropes for ULS (K2 NEN3508)

$$F_{rope.O.20_z} := k_{b.O} \cdot \max\left(\widehat{m_{20}^{z}}\right) \cdot g = \begin{bmatrix} 6962.7 \\ 8703.4 \\ 17406.8 \\ 34813.6 \end{bmatrix} kN \qquad F_{rope.O.15_z} := k_{b.O} \cdot \max\left(\widehat{m_{15}^{z}}\right) \cdot g = \begin{bmatrix} 5222.0 \\ 6527.6 \\ 13055.1 \\ 26110.2 \end{bmatrix} kN$$

Assume steel wire rope with maximum diameter 60 mm and tensile strength 1960 MPa

$F_{br} := 2500 \cdot kN$ Minimum breaking force

Number of wire ropes

$$n_{rope.20} := ceil\left(\frac{F_{rope.O.20}}{F_{br}}\right) = \begin{bmatrix} 3.0 \\ 4.0 \\ 7.0 \\ 14.0 \end{bmatrix} \qquad n_{rope.15} := ceil\left(\frac{F_{rope.O.15}}{F_{br}}\right) = \begin{bmatrix} 3.0 \\ 3.0 \\ 6.0 \\ 11.0 \end{bmatrix}$$

$s_g := \frac{20 \cdot m}{2} \cdot \sqrt{3} = 17.3\ m$

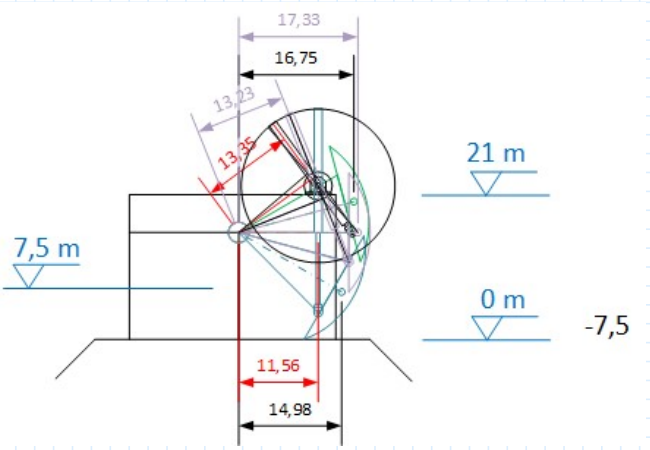
$r_{gs} := s_g = 17.3\ m$

$s_{gs} := 2 \cdot r_{gs} \cdot \sin\left(\frac{\alpha_g}{2}\right) = 24.5\ m$

$\alpha_{gs} := 2 \cdot \operatorname{asin}\left(\frac{s_g}{2 \cdot r_{gs}}\right) = 60.0\ deg$

$L_g := \begin{bmatrix} 16.75 \\ 17.33 \\ 14.98 \end{bmatrix} \cdot m \quad L_c := \begin{bmatrix} 13.35 \\ 13.23 \\ 11.56 \end{bmatrix} \cdot m$

$X_g := \frac{\overrightarrow{L_g}}{L_c}$



$F_{h.max} := m_{20} \cdot g \cdot \max(X_g) = \begin{bmatrix} 2569.1 \\ 3211.4 \\ 6422.9 \\ 12845.7 \end{bmatrix} kN$

Maximum cylinder load

$\gamma_U := 1.5$ Loadfactor ULS

$F_{cil} := \frac{1}{2} \cdot \gamma_U \cdot F_{h.max} = \begin{bmatrix} 1926.9 \\ 2408.6 \\ 4817.2 \\ 9634.3 \end{bmatrix} kN$

Assume maximum cilinder pressure of 200 bar

$p_{cil} := 200 \cdot bar$ $\varphi := 2$

$d_{z15} := \sqrt{\frac{4 \cdot F_{cil}}{\pi \cdot p_{cil} \cdot \left(1 - \frac{1}{\varphi}\right)}} = \begin{bmatrix} 495.3 \\ 553.8 \\ 783.2 \\ 1107.6 \end{bmatrix} mm \quad d_{s15} := \frac{1}{\sqrt{\varphi}} \cdot d_{z15} = \begin{bmatrix} 350.2 \\ 391.6 \\ 553.8 \\ 783.2 \end{bmatrix} mm \quad p := \frac{F_{cil}}{\frac{\pi}{4} \cdot \left(d_{z15}^2 - d_{s15}^2\right)} = \begin{bmatrix} 200.0 \\ 200.0 \\ 200.0 \\ 200.0 \end{bmatrix} bar$

With only one cylinder

$F_{cil} := \gamma_U \cdot F_{h.max} = \begin{bmatrix} 3853.7 \\ 4817.2 \\ 9634.3 \\ 19268.6 \end{bmatrix} kN$

Assume maximum cilinder pressure of 200 bar

$p_{cil} := 200 \cdot bar$

$d_{z15} := \sqrt{\frac{4 \cdot F_{cil}}{\pi \cdot p_{cil} \cdot \left(1 - \frac{1}{\varphi}\right)}} = \begin{bmatrix} 700.5 \\ 783.2 \\ 1107.6 \\ 1566.3 \end{bmatrix} mm \quad d_{s15} := \frac{1}{\sqrt{\varphi}} \cdot d_{z15} = \begin{bmatrix} 495.3 \\ 553.8 \\ 783.2 \\ 1107.6 \end{bmatrix} mm \quad p := \frac{F_{cil}}{\frac{\pi}{4} \cdot \left(d_{z15}^2 - d_{s15}^2\right)} = \begin{bmatrix} 200.0 \\ 200.0 \\ 200.0 \\ 200.0 \end{bmatrix} bar$

Braking power

Assume closing time 30 minutes

$t := 30 \cdot min$ $sl := 10 \cdot m$

$v_{15} := \frac{sl}{t} = 5.6\ \frac{mm}{s}$

$$P_{15.br} := 2 \cdot F_{cil} \cdot v_{15} = \begin{bmatrix} 42.8 \\ 53.5 \\ 107.0 \\ 214.1 \end{bmatrix} \text{kW} \qquad W = \begin{bmatrix} 20.0 \\ 25.0 \\ 50.0 \\ 100.0 \end{bmatrix} \text{m}$$

Opening power

$\eta := 0.6$

$$P_{15.o} := \frac{P_{15.br}}{\eta} = \begin{bmatrix} 71.4 \\ 89.2 \\ 178.4 \\ 356.8 \end{bmatrix} \text{kW}$$

hinged flap

$$W = \begin{bmatrix} 20.0 \\ 25.0 \\ 50.0 \\ 100.0 \end{bmatrix} \text{m} \qquad m_{20} = \begin{bmatrix} 200000.0 \\ 250000.0 \\ 500000.0 \\ 1000000.0 \end{bmatrix} \text{kg} \qquad m_{15} = \begin{bmatrix} 150000.0 \\ 187500.0 \\ 375000.0 \\ 750000.0 \end{bmatrix} \text{kg}$$

wet height of the gate

$$dH = \begin{bmatrix} 0.0 \\ 0.6 \\ 1.2 \\ 1.8 \\ 2.4 \\ 3.0 \\ 3.6 \\ 4.2 \\ 4.8 \\ 5.4 \\ 6.0 \end{bmatrix} \text{m}$$

$$\alpha_g := \begin{bmatrix} 0 \\ 15 \\ 30 \\ 45 \\ 60 \\ 75 \\ 90 \end{bmatrix} \cdot \text{deg} \qquad x_{G20} := \begin{bmatrix} 11.49 \\ 10.71 \\ 9.61 \\ 7.85 \\ 5.56 \\ 2.88 \\ 0 \end{bmatrix} \cdot \text{m} \qquad la_{20} := \begin{bmatrix} 19.41 \\ 19.32 \\ 18.58 \\ 17.45 \\ 16.05 \\ 14.35 \\ 12.42 \end{bmatrix} \cdot \text{m} \qquad wh_{20} := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 4.85 \\ 8.49 \\ 10.91 \\ 12 \end{bmatrix} \cdot \text{m}$$

$$x_{G15} := \begin{bmatrix} 8.82 \\ 8.53 \\ 7.66 \\ 6.26 \\ 4.43 \\ 2.30 \\ 0 \end{bmatrix} \cdot \text{m} \qquad la_{15} := \begin{bmatrix} 14.45 \\ 14.47 \\ 13.93 \\ 13.02 \\ 11.86 \\ 10.48 \\ 8.91 \end{bmatrix} \cdot \text{m} \qquad wh_{15} := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 1.32 \\ 4.14 \\ 6.07 \\ 7 \end{bmatrix} \cdot \text{m}$$

$k := 1 .. \text{rows}(x_{G20})$

$\Delta H_{20} := 0.2 \cdot \text{m}$

$$F_{wr20_{k,z}} := \frac{m_{20_z} \cdot g \cdot x_{G20_k}}{la_{20_k}} + \text{if}\left(\alpha_{g_k} \neq 0, \frac{\Delta H_{20} \cdot \rho_w \cdot g \cdot W_z \cdot wh_{20_k}}{\sin\left(\alpha_{g_k}\right)}, 0\right)$$

$\Delta H_{15} := 0.35 \cdot \text{m}$

$$F_{wr15_{k,z}} := \frac{m_{15_z} \cdot g \cdot x_{G15_k}}{la_{15_k}} + \text{if}\left(\alpha_{g_k} \neq 0, \frac{\Delta H_{15} \cdot \rho_w \cdot g \cdot W_z \cdot wh_{15_k}}{\sin\left(\alpha_{g_k}\right)}, 0\right)$$

$\Delta H_{20} = 0.2\ m$ $W^T = [20.0\ 25.0\ 50.0\ 100.0]\ m$

$$\alpha_g = \begin{bmatrix} 0.0 \\ 15.0 \\ 30.0 \\ 45.0 \\ 60.0 \\ 75.0 \\ 90.0 \end{bmatrix} deg \quad F_{wr20} = \begin{bmatrix} 1161 & 1451 & 2903 & 5805 \\ 1087 & 1359 & 2718 & 5436 \\ 1014 & 1268 & 2536 & 5072 \\ 1159 & 1449 & 2899 & 5797 \\ 1076 & 1344 & 2689 & 5378 \\ 850 & 1062 & 2125 & 4250 \\ 485 & 606 & 1212 & 2424 \end{bmatrix} kN \quad wh_{20} = \begin{bmatrix} 0.0 \\ 0.0 \\ 0.0 \\ 4.9 \\ 8.5 \\ 10.9 \\ 12.0 \end{bmatrix} m$$

$\Delta H_{15} = 0.35\ m$ $W^T = [20.0\ 25.0\ 50.0\ 100.0]\ m$

$$\alpha_g = \begin{bmatrix} 0.0 \\ 15.0 \\ 30.0 \\ 45.0 \\ 60.0 \\ 75.0 \\ 90.0 \end{bmatrix} deg \quad F_{wr15} = \begin{bmatrix} 898 & 1122 & 2245 & 4489 \\ 867 & 1084 & 2168 & 4336 \\ 809 & 1011 & 2022 & 4044 \\ 839 & 1049 & 2098 & 4196 \\ 887 & 1109 & 2219 & 4437 \\ 767 & 959 & 1918 & 3836 \\ 495 & 619 & 1237 & 2475 \end{bmatrix} kN \quad wh_{15} = \begin{bmatrix} 0.0 \\ 0.0 \\ 0.0 \\ 1.3 \\ 4.1 \\ 6.1 \\ 7.0 \end{bmatrix} m$$

Assume a wire rope will be used

$k_{b.0} := 3.55$ Service factor wire ropes for ULS (K2 NEN3508)

$$F_{rope.0.20} := \frac{k_{b.0} \cdot m_{20} \cdot g}{2 \cdot 2} = \begin{bmatrix} 1740.7 \\ 2175.9 \\ 4351.7 \\ 8703.4 \end{bmatrix} kN \qquad F_{rope.0.15} := \frac{k_{b.0} \cdot m_{15} \cdot g}{2 \cdot 2} = \begin{bmatrix} 1305.5 \\ 1631.9 \\ 3263.8 \\ 6527.6 \end{bmatrix} kN$$

Assume steel wire rope with maximum diameter 60 mm and tensile strength 1960 MPa

$F_{br} := 2500 \cdot kN$ Minimum breaking force

Number of wire ropes per side of the gate

$$n_{rope.20} := ceil\left(\frac{F_{rope.0.20}}{F_{br}}\right) = \begin{bmatrix} 1.0 \\ 1.0 \\ 2.0 \\ 4.0 \end{bmatrix} \qquad n_{rope.15} := ceil\left(\frac{F_{rope.0.15}}{F_{br}}\right) = \begin{bmatrix} 1.0 \\ 1.0 \\ 2.0 \\ 3.0 \end{bmatrix}$$

Braking power

Assume closing time 30 minutes

$t := 30 \cdot min$ $\Delta L_{rope} := 30.6 \cdot m - 5.3 \cdot m$

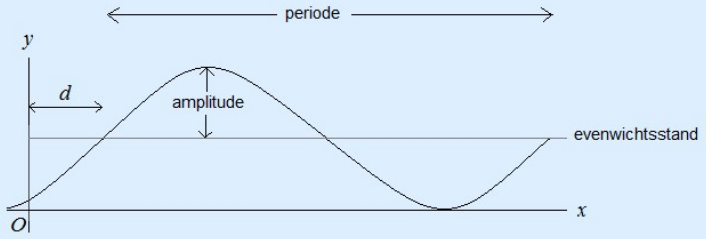
$$v_{20} := \frac{\Delta L_{rope}}{t} = 14.1\ \frac{mm}{s} \qquad v_{15} := \frac{\Delta L_{rope}}{t} = 14.1\ \frac{mm}{s}$$

$$P_{20} := v_{20} \cdot \left(\overset{\sim}{F_{wr20}}^{\langle 1 \rangle}\right)^T = \begin{bmatrix} 16.3 \\ 20.4 \\ 40.8 \\ 81.6 \end{bmatrix} kW \qquad P_{15} := v_{15} \cdot \left(\overset{\sim}{F_{wr15}}^{\langle 1 \rangle}\right)^T = \begin{bmatrix} 12.6 \\ 15.8 \\ 31.6 \\ 63.1 \end{bmatrix} kW$$

De algemene formule van een sinusoïde is:

$$y = a + b\ \sin(c(x - d))$$

Hierin is:
a de evenwichtsstand
b de amplitude
c 2π/periode
d de x-coördinaat van een punt waar de grafiek stijgend de evenwichtsstand snijdt



$T := 0, 0.1 .. 24$

$b := 3 \cdot m$ $a := 0 \cdot m$ $c := \frac{2 \cdot \pi}{12}$ $d := 0$ $x := T$

$y(x) := a + b \cdot \sin(c \cdot (x - d))$

$x_g := 10$ Guess value

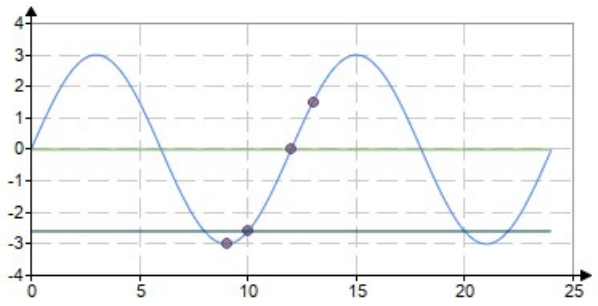
$x_0 := \mathbf{root}(y(x_g), x_g) = 12.0$

$y(x_0 + 1) = 1.5\ m$ $\Delta H := y(x_0 + 1) - y(x_0) = 1.5\ m$

$x_g := 8$ Guess value

$gf(x) := \frac{d}{dx} y(x)$ $x_d := \mathbf{root}(gf(x_g), x_g) = 9.0$

$y(x_d + 1) = -2.6\ m$ $\Delta H := y(x_d + 1) - y(x_d) = 0.4\ m$



Probability of failure

$\beta := 3.8$ Reliability index

$\mu := 50$ Reference period (year)

$\sigma := \frac{\mu}{\beta} = 13.2$ Standard deviation

$P := pnorm(\mu - \beta \cdot \sigma, \mu, \sigma) = 72.3 \cdot 10^{-6}$ Failure probability over the considered lifetime

$P_f := \frac{1}{\mu} \cdot pnorm(\mu - \beta \cdot \sigma, \mu, \sigma) \cdot \frac{1}{yr} = (1.4 \cdot 10^{-6}) \frac{1}{yr}$ Failure rate

$N := 2$ Number of components

$P_t := N \cdot P = 144.7 \cdot 10^{-6}$ System failure probability

$P_{f.t.yr} := N \cdot P_f = (2.9 \cdot 10^{-6}) \frac{1}{yr}$ System failure rate

$P_{f.t.hr} := P_{f.t.yr} = (330.1 \cdot 10^{-12}) \frac{1}{hr}$ System failure rate

Roller bearing (Weibull)

$\beta := 1.3$ Weibull shape factor

$T := 50 \cdot yr$ Lifespan

$B_{10} := 0.9$ L10 reliability

$\delta := 0 \cdot yr$ Failure-free life

$\eta := \frac{T - \delta}{(-\ln(B_{10}))^{\frac{1}{\beta}}} = 282.3\ yr$ Virtual lifespan

$R(t) := e^{-\left(\frac{t-\delta}{\eta}\right)^{\beta}}$ Weibull

$\lambda_{av} := \frac{1 - R(T)}{\int_0^T R(t)\, dt}$ Failure frequency

$\lambda_{av} = (238.7 \cdot 10^{-9}) \frac{1}{hr}$

$MTBF := \lambda_{av}^{-1} = 4188716.9\ hr$ $MTBF = 477.8\ yr$

Data

Event 1

$\lambda_1 := 1.1 \cdot 10^{-4} \cdot \frac{1}{hr}$

$\theta_1 := 12 \cdot hr$

$MTBF_1 := \frac{1}{\lambda_1} = 9090.9\ hr$

$A_1 := \frac{MTBF_1}{MTBF_1 + \theta_1} = 1.0$

$Q_1 := 1 - A_1 = 1.32 \cdot 10^{-3}$

$w_1 := \frac{1}{MTBF_1 + \theta_1} = (1.10 \cdot 10^{-4}) \frac{1}{hr}$

$Q_{and} := Q_1 \cdot Q_2 = 2.19 \cdot 10^{-7}$

$Q_{and} := 1 - (A_1 + A_2 - A_1 \cdot A_2) = 2.19 \cdot 10^{-7}$

$w_{and} := w_1 \cdot w_2 \cdot (\theta_1 + \theta_2) = (4.57 \cdot 10^{-8}) \frac{1}{hr}$

Event 2

$\lambda_2 := 2.08 \cdot 10^{-5} \cdot \frac{1}{hr}$

$\theta_2 := 8 \cdot hr$

$MTBF_2 := \frac{1}{\lambda_2} = 48076.9\ hr$

$A_2 := \frac{MTBF_2}{MTBF_2 + \theta_2} = 1.0$

$Q_2 := 1 - A_2 = 1.66 \cdot 10^{-4}$

$w_2 := \frac{1}{MTBF_2 + \theta_2} = (2.08 \cdot 10^{-5}) \frac{1}{hr}$

$Q_{or} := Q_1 + Q_2 - Q_1 \cdot Q_2 = 1.48 \cdot 10^{-3}$

$w_{or} := w_1 + w_2 = (1.31 \cdot 10^{-4}) \frac{1}{hr}$

Failure rate (component data)

Repair time (MTTR)

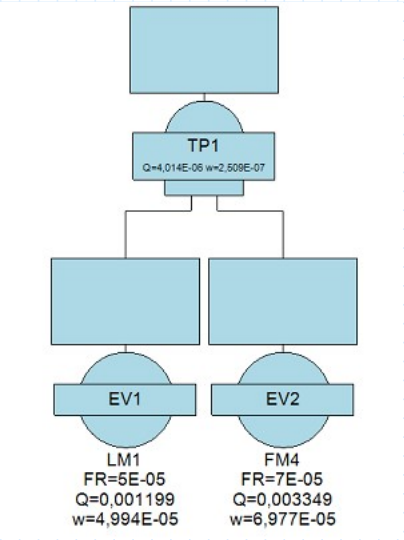
Mean Time Between Failure

Availiability

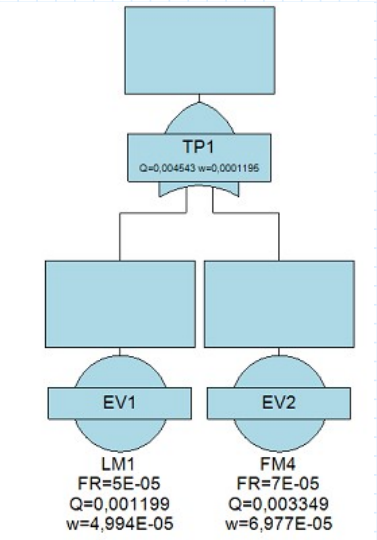
Unavailiability

Failure rate

AND-gate



OR-gate



$w := 7.713 \cdot 10^{-6} \cdot \frac{1}{hr}$

$Q := 0.001479$

$MTTF := \frac{1}{w} = 129651.2\ hr$

$MTTR := \frac{Q}{w} = 191.8\ hr$

$\lambda := \frac{1}{MTTF + -MTTR} = (7.72 \cdot 10^{-6}) \frac{1}{hr}$

Contact velocity vertical lift gate at free fall

$L := 100\ m$ $H := 20\ m$ Height and lenght of the considered gate

$h_0 := 14\ m$ $h_{water} := 0\ m$ Inatial height of the gate above waterlevel and waterlevel

$w := 500\ \frac{kg}{m^2}$ Specefic mass of the gate

$m_{gate} := w \cdot L \cdot H = (1.0 \cdot 10^6)\ kg$ Mass of the gate

$G_{gate} := w \cdot L \cdot H \cdot g = (9.8 \cdot 10^3)\ kN$ Weight of the gate

Starting point

$v_0 := 0\ \frac{m}{s}$ Initeal velocity of the gate

$dh := h_0 - h_{water}$

$E_{k0} := \frac{1}{2} \cdot m_{gate} \cdot v_0^2 = 0.0\ J$ Kinetic energy

$E_p := G_{gate} \cdot dh = 137293100.0\ J$ Potential energy

$v := \sqrt{\frac{2 \cdot E_p}{m_{gate}}} = 16.6\ \frac{m}{s}$ $v = 59.7\ \frac{km}{hr}$ Contact velocity water

$\rho_{st} := 7850 \cdot \frac{kg}{m^3}$ $\rho_w := 1030 \cdot \frac{kg}{m^3}$

$h_b := -12 \cdot m$ $m_w := m_{gate} \cdot \left(\frac{\rho_w}{\rho_{st}}\right)$

$dh_2 := h_{water} - h_b$

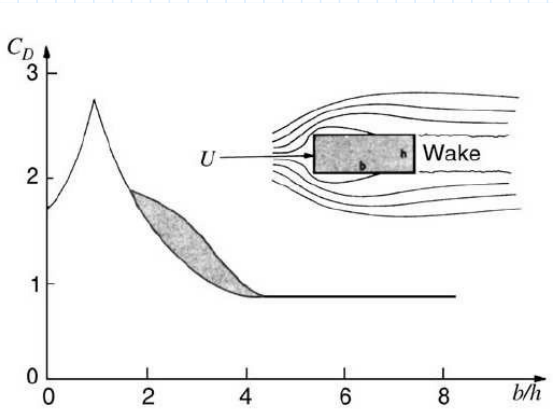
$E_{p2} := m_{gate} \cdot g \cdot dh_2$

$W := m_w \cdot g \cdot \frac{dh_2}{2}$ Buoyancy, half height of the gate

$t_{gate} := 1 \cdot m$ Gate thickness

$A := L \cdot t_{gate}$

$C_D := 1$ Drag coëfficiënt when of the gate in the water

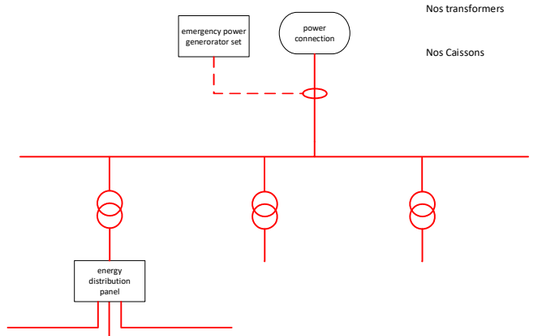


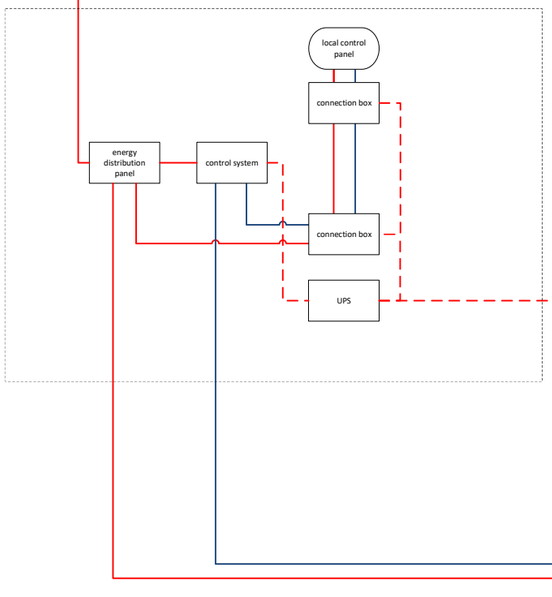
$E_k := \frac{1}{2} \cdot m_{gate} \cdot v^2$

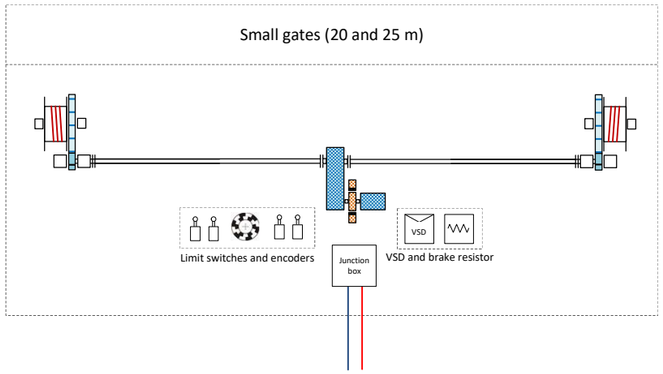
$v_e := \sqrt{\frac{E_{p2} + E_k - W}{\left(\frac{1}{2} \cdot m_{gate} + \frac{1}{2} \cdot \rho_w \cdot C_D \cdot A \cdot dh_2\right)}} = 14.9\ \frac{m}{s}$ $v_e = 53.5\ \frac{km}{hr}$

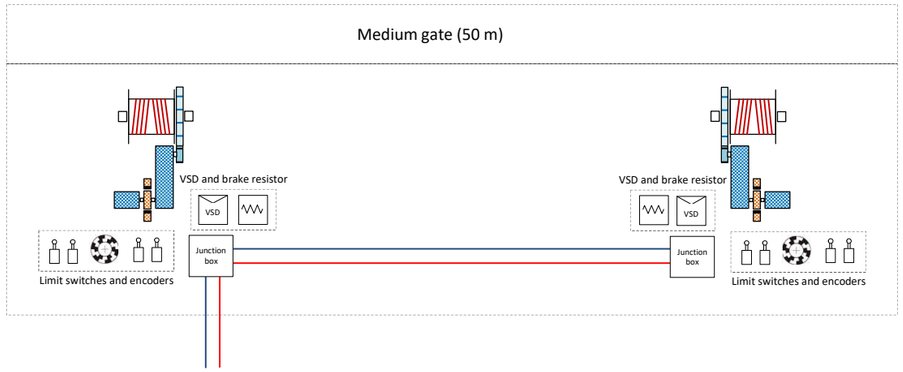
A11 PRINCIPLE OF HYDRAULIC OPERATING MECHANISM

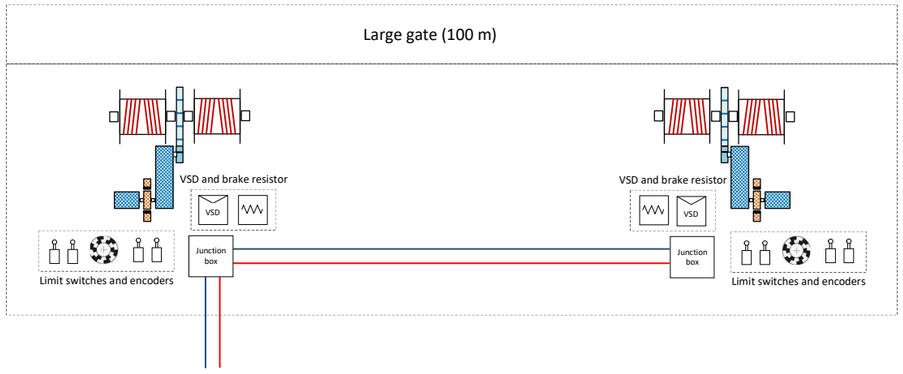
	Dam					
	1 km	5 km	10 km	15 km	20 km	25 km
Nos transformers	1	3	7	10	14	17
Nos Caissons	Transformer					
	14					





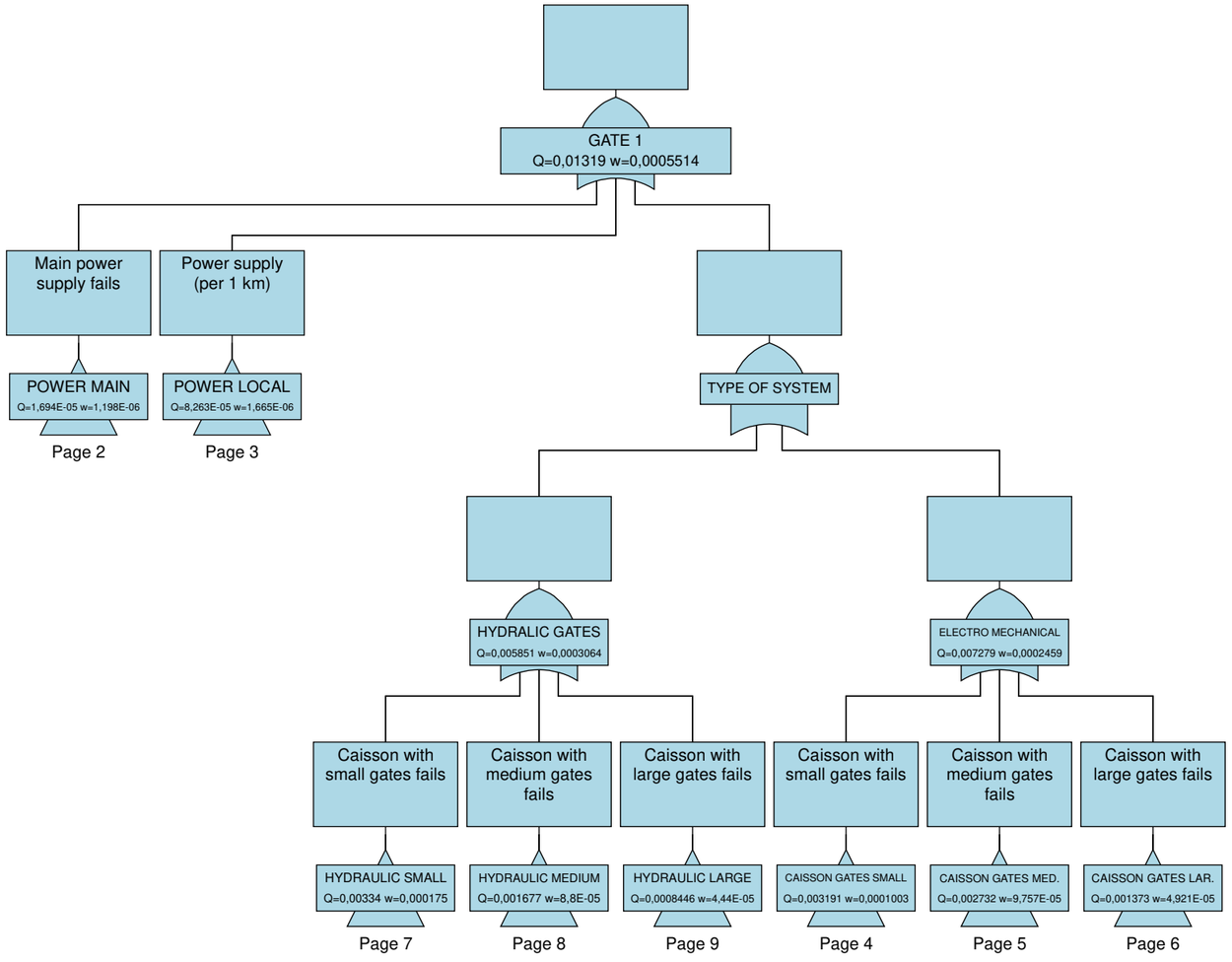


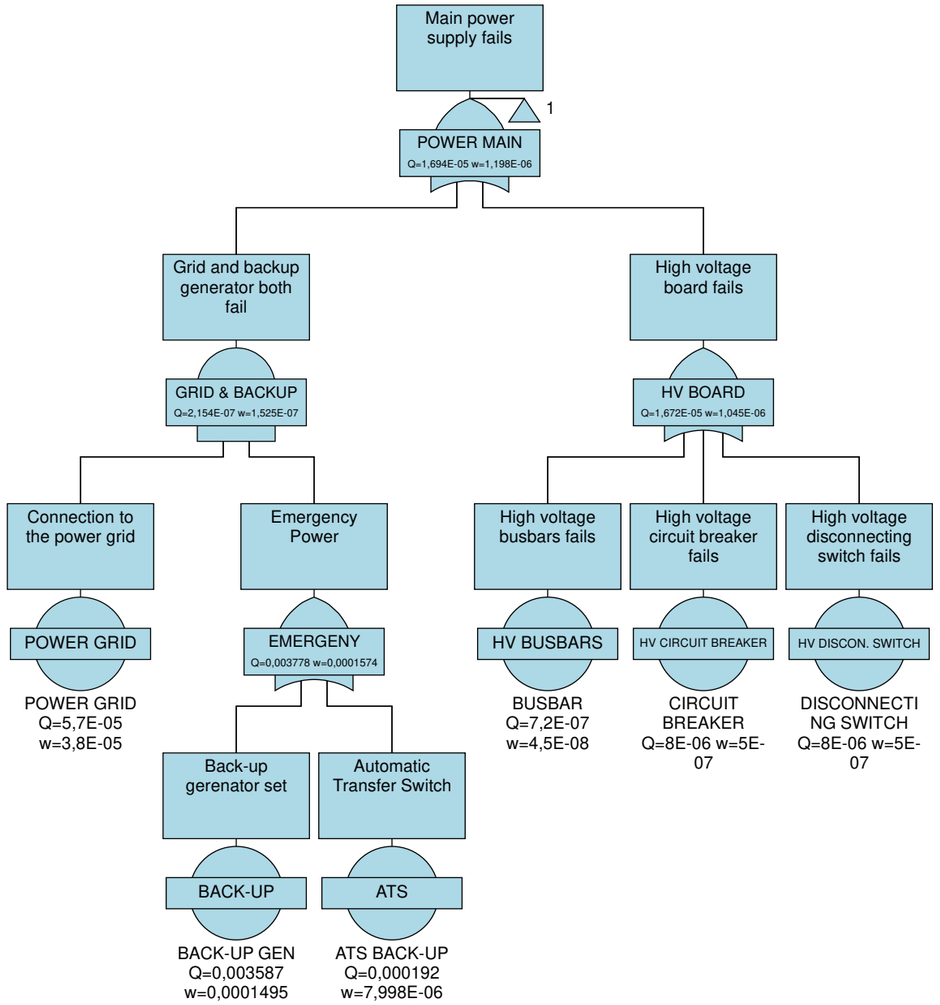


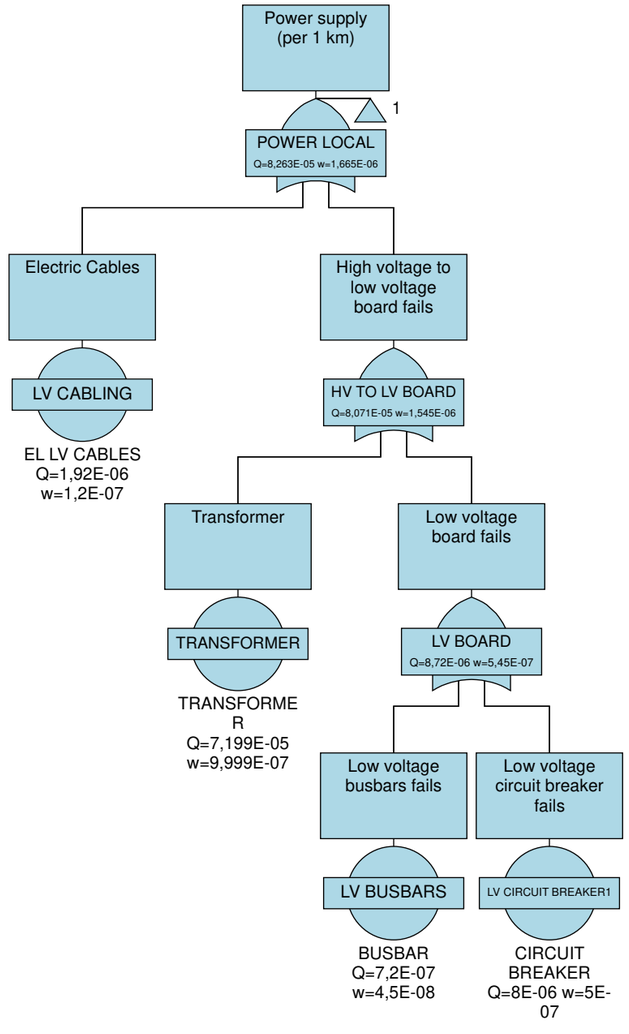


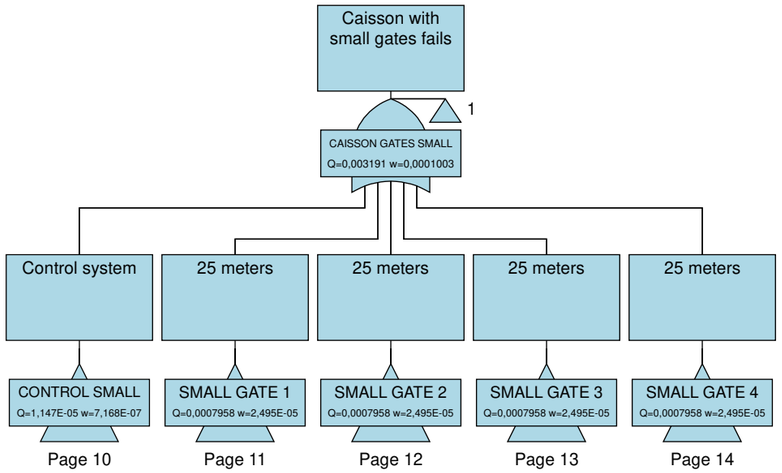
Opdrachtgever:			
Project:			
Omschrijving:		Projectnummer:	
		In bladen, blad nr.	
		Formaat:	
		Schaal:	
Getekend:		Paraaf: ...	d.d.
Gecontroleerd:		Paraaf: ...	d.d.
Vrijgegeven:		Paraaf: ...	d.d.
Status:		Versie:	Tekeningnr.:

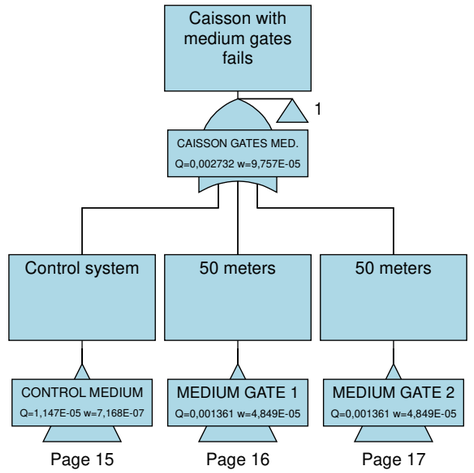
A12 INDICATIVE FAULT TREE ANALYSIS

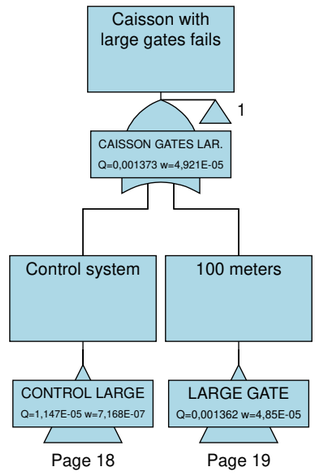

GATE 1
Q=0,01319 w=0,0005514
Main power supply fails
Power supply (per 1 km)
POWER MAIN
Q=1,694E-05 w=1,198E-06
POWER LOCAL
Q=8,263E-05 w=1,665E-06
Page 2
Page 3
TYPE OF SYSTEM
HYDRALIC GATES
Q=0,005851 w=0,0003064
ELECTRO MECHANICAL
Q=0,007279 w=0,0002459
Caisson with small gates fails
Caisson with medium gates fails
Caisson with large gates fails
Caisson with small gates fails
Caisson with medium gates fails
Caisson with large gates fails
HYDRAULIC SMALL
Q=0,00334 w=0,000175
HYDRAULIC MEDIUM
Q=0,001677 w=8,8E-05
HYDRAULIC LARGE
Q=0,0008446 w=4,44E-05
CAISSON GATES SMALL
Q=0,003191 w=0,0001003
CAISSON GATES MED.
Q=0,002732 w=9,757E-05
CAISSON GATES LAR.
Q=0,001373 w=4,921E-05
Page 7
Page 8
Page 9
Page 4
Page 5
Page 6

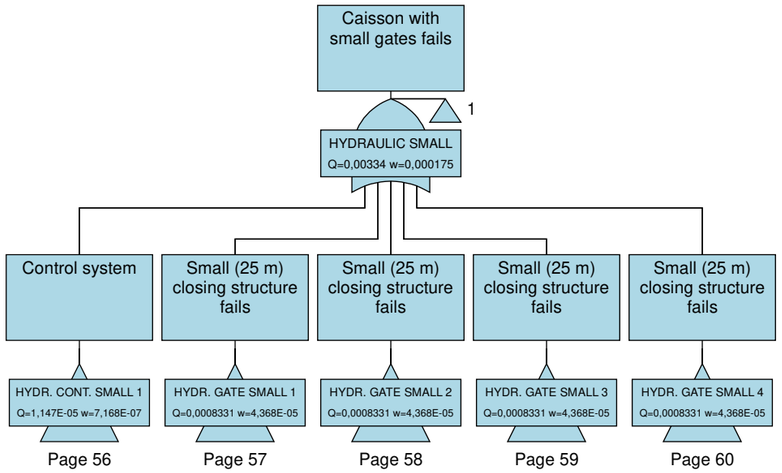

Main power supply fails
1
POWER MAIN
Q=1,694E-05 w=1,198E-06
Grid and backup generator both fail
High voltage board fails
GRID & BACKUP
Q=2,154E-07 w=1,525E-07
HV BOARD
Q=1,672E-05 w=1,045E-06
Connection to the power grid
Emergency Power
High voltage busbars fails
High voltage circuit breaker fails
High voltage disconnecting switch fails
POWER GRID
EMERGENY
Q=0,003778 w=0,0001574
HV BUSBARS
HV CIRCUIT BREAKER
HV DISCON. SWITCH
POWER GRID
Q=5,7E-05
w=3,8E-05
BUSBAR
Q=7,2E-07
w=4,5E-08
CIRCUIT BREAKER
Q=8E-06 w=5E-07
DISCONNECTING SWITCH
Q=8E-06 w=5E-07
Back-up gerenator set
Automatic Transfer Switch
BACK-UP
ATS
BACK-UP GEN
Q=0,003587
w=0,0001495
ATS BACK-UP
Q=0,000192
w=7,998E-06

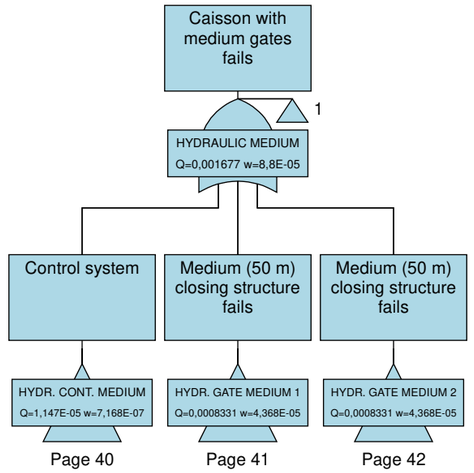

Power supply
(per 1 km)
1
POWER LOCAL
Q=8,263E-05 w=1,665E-06
Electric Cables
High voltage to low voltage board fails
LV CABLING
HV TO LV BOARD
Q=8,071E-05 w=1,545E-06
EL LV CABLES
Q=1,92E-06
w=1,2E-07
Transformer
Low voltage board fails
TRANSFORMER
LV BOARD
Q=8,72E-06 w=5,45E-07
TRANSFORME
R
Q=7,199E-05
w=9,999E-07
Low voltage busbars fails
Low voltage circuit breaker fails
LV BUSBARS
LV CIRCUIT BREAKER1
BUSBAR
Q=7,2E-07
w=4,5E-08
CIRCUIT
BREAKER
Q=8E-06 w=5E-
07

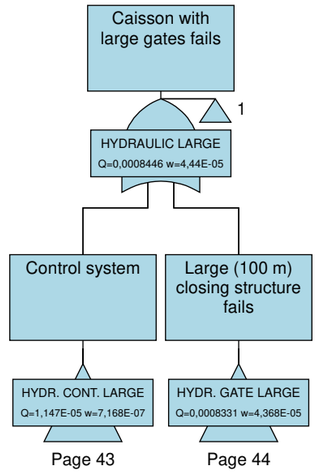

Caisson with small gates fails
1
CAISSON GATES SMALL
Q=0,003191 w=0,0001003
Control system
25 meters
25 meters
25 meters
25 meters
CONTROL SMALL
Q=1,147E-05 w=7,168E-07
SMALL GATE 1
Q=0,0007958 w=2,495E-05
SMALL GATE 2
Q=0,0007958 w=2,495E-05
SMALL GATE 3
Q=0,0007958 w=2,495E-05
SMALL GATE 4
Q=0,0007958 w=2,495E-05
Page 10
Page 11
Page 12
Page 13
Page 14

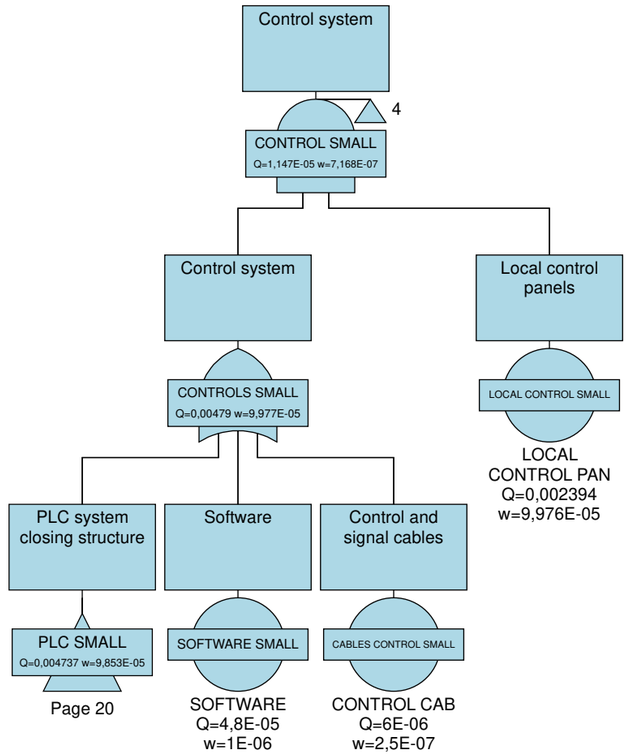

Caisson with medium gates fails
1
CAISSON GATES MED.
Q=0,002732 w=9,757E-05
Control system
50 meters
50 meters
CONTROL MEDIUM
Q=1,147E-05 w=7,168E-07
MEDIUM GATE 1
Q=0,001361 w=4,849E-05
MEDIUM GATE 2
Q=0,001361 w=4,849E-05
Page 15
Page 16
Page 17

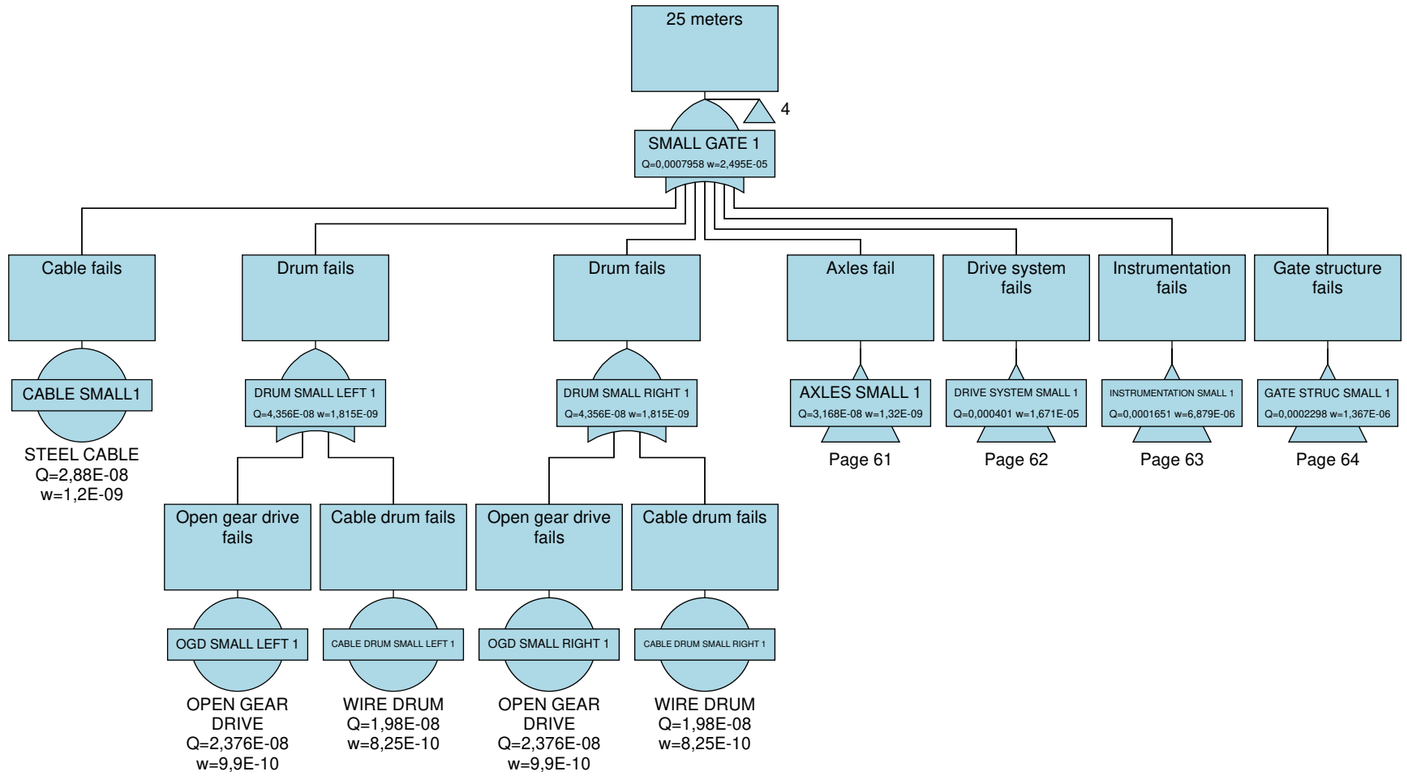

Caisson with large gates fails
1
CAISSON GATES LAR.
Q=0,001373 w=4,921E-05
Control system
100 meters
CONTROL LARGE
Q=1,147E-05 w=7,168E-07
LARGE GATE
Q=0,001362 w=4,85E-05
Page 18
Page 19

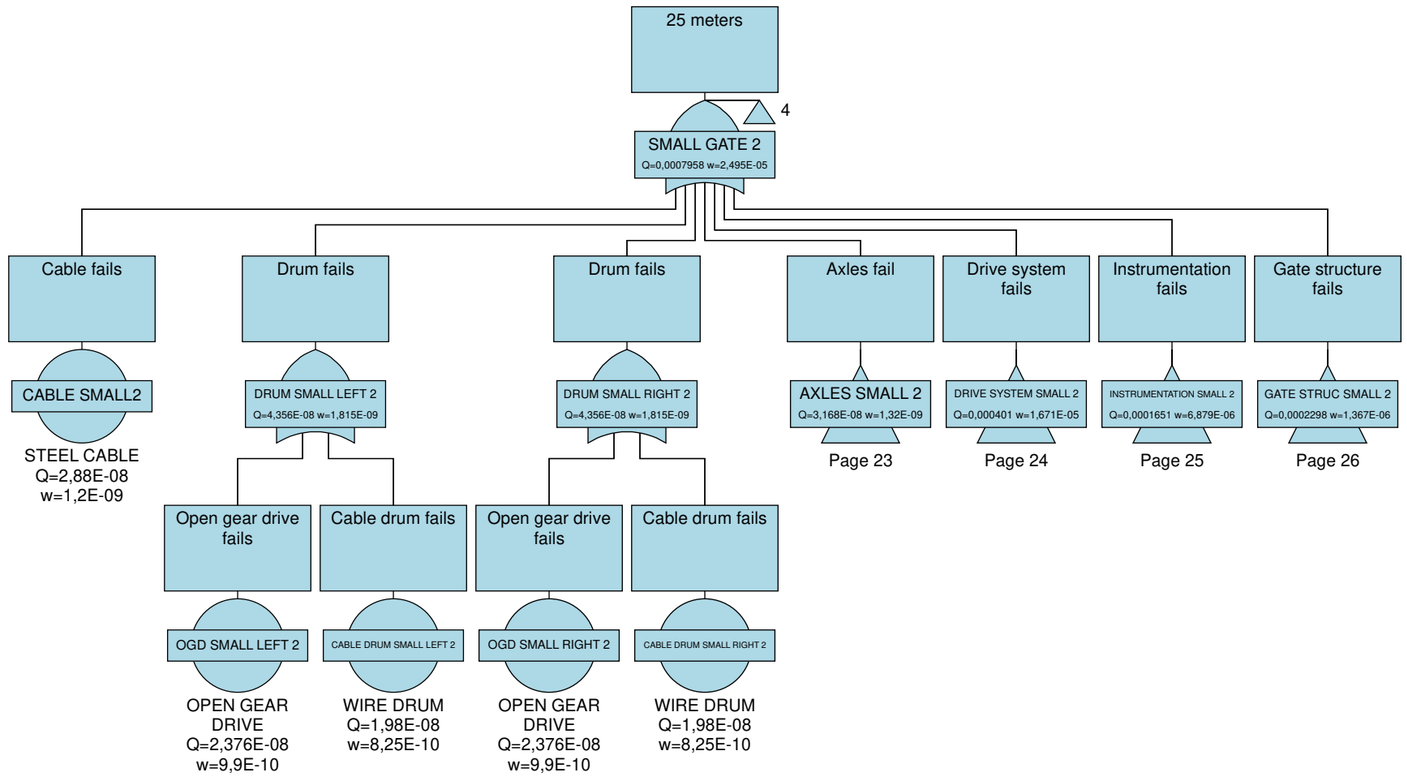

Caisson with small gates fails
1
HYDRAULIC SMALL
Q=0,00334 w=0,000175
Control system
Small (25 m) closing structure fails
Small (25 m) closing structure fails
Small (25 m) closing structure fails
Small (25 m) closing structure fails
HYDR. CONT. SMALL 1
Q=1,147E-05 w=7,168E-07
HYDR. GATE SMALL 1
Q=0,0008331 w=4,368E-05
HYDR. GATE SMALL 2
Q=0,0008331 w=4,368E-05
HYDR. GATE SMALL 3
Q=0,0008331 w=4,368E-05
HYDR. GATE SMALL 4
Q=0,0008331 w=4,368E-05
Page 56
Page 57
Page 58
Page 59
Page 60

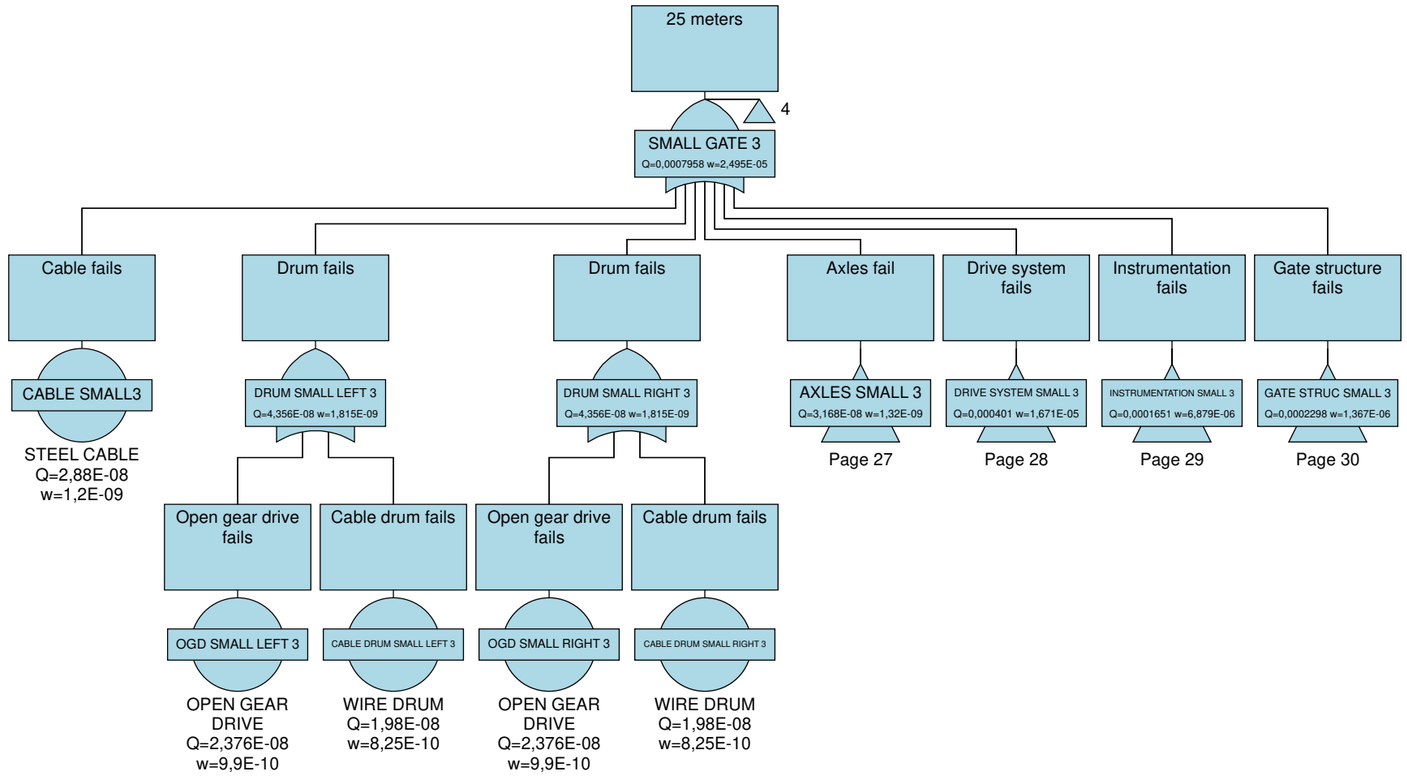

Caisson with medium gates fails
1
HYDRAULIC MEDIUM
Q=0,001677 w=8,8E-05
Control system
Medium (50 m) closing structure fails
Medium (50 m) closing structure fails
HYDR. CONT. MEDIUM
Q=1,147E-05 w=7,168E-07
HYDR. GATE MEDIUM 1
Q=0,0008331 w=4,368E-05
HYDR. GATE MEDIUM 2
Q=0,0008331 w=4,368E-05
Page 40
Page 41
Page 42

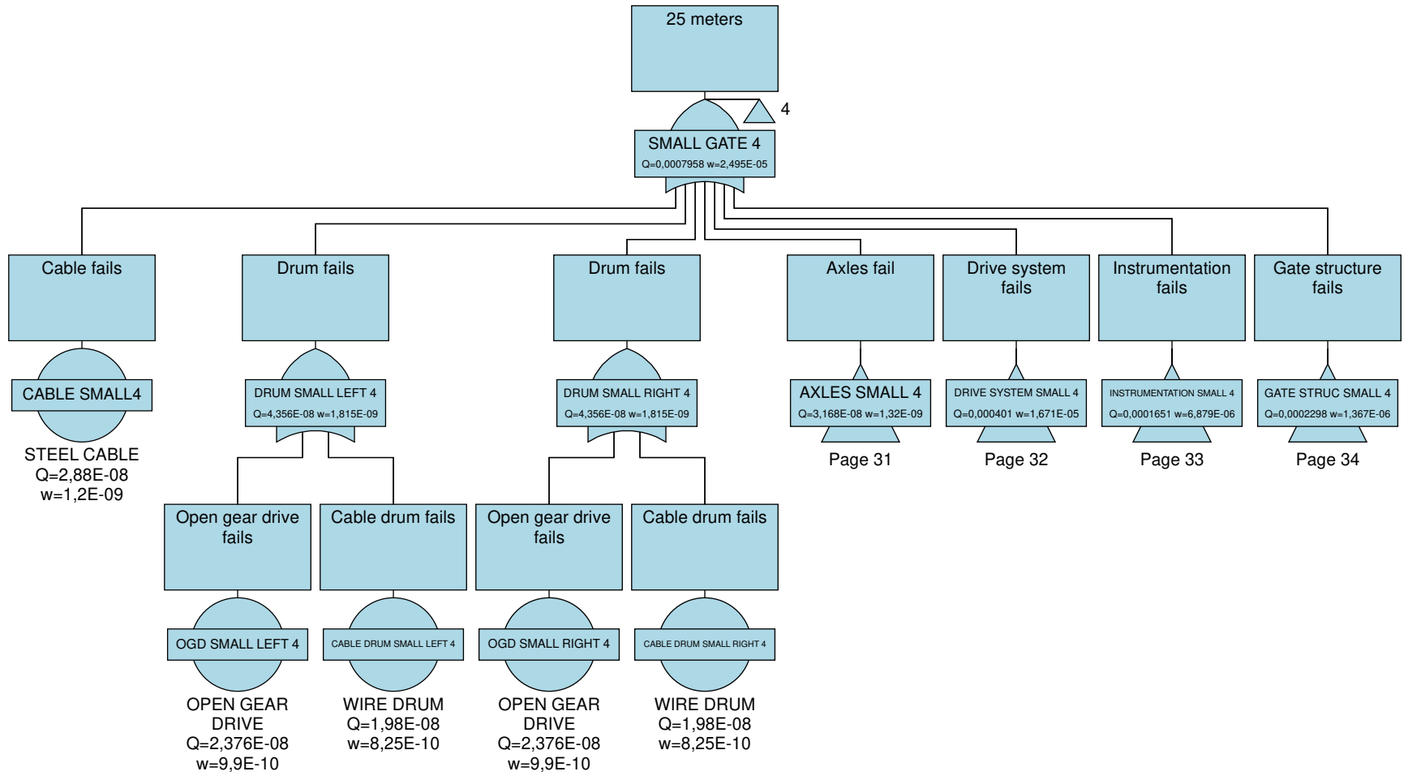

Caisson with large gates fails
1
HYDRAULIC LARGE
Q=0,0008446 w=4,44E-05
Control system
Large (100 m) closing structure fails
HYDR. CONT. LARGE
Q=1,147E-05 w=7,168E-07
HYDR. GATE LARGE
Q=0,0008331 w=4,368E-05
Page 43
Page 44

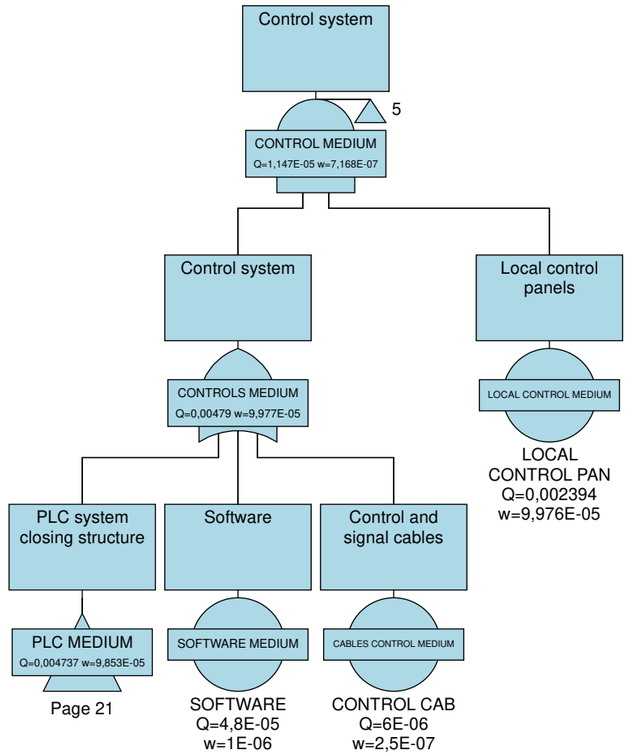

Control system
4
CONTROL SMALL
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
CONTROLS SMALL
Q=0,00479 w=9,977E-05
LOCAL CONTROL SMALL
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC SMALL
Q=0,004737 w=9,853E-05
SOFTWARE SMALL
CABLES CONTROL SMALL
Page 20
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

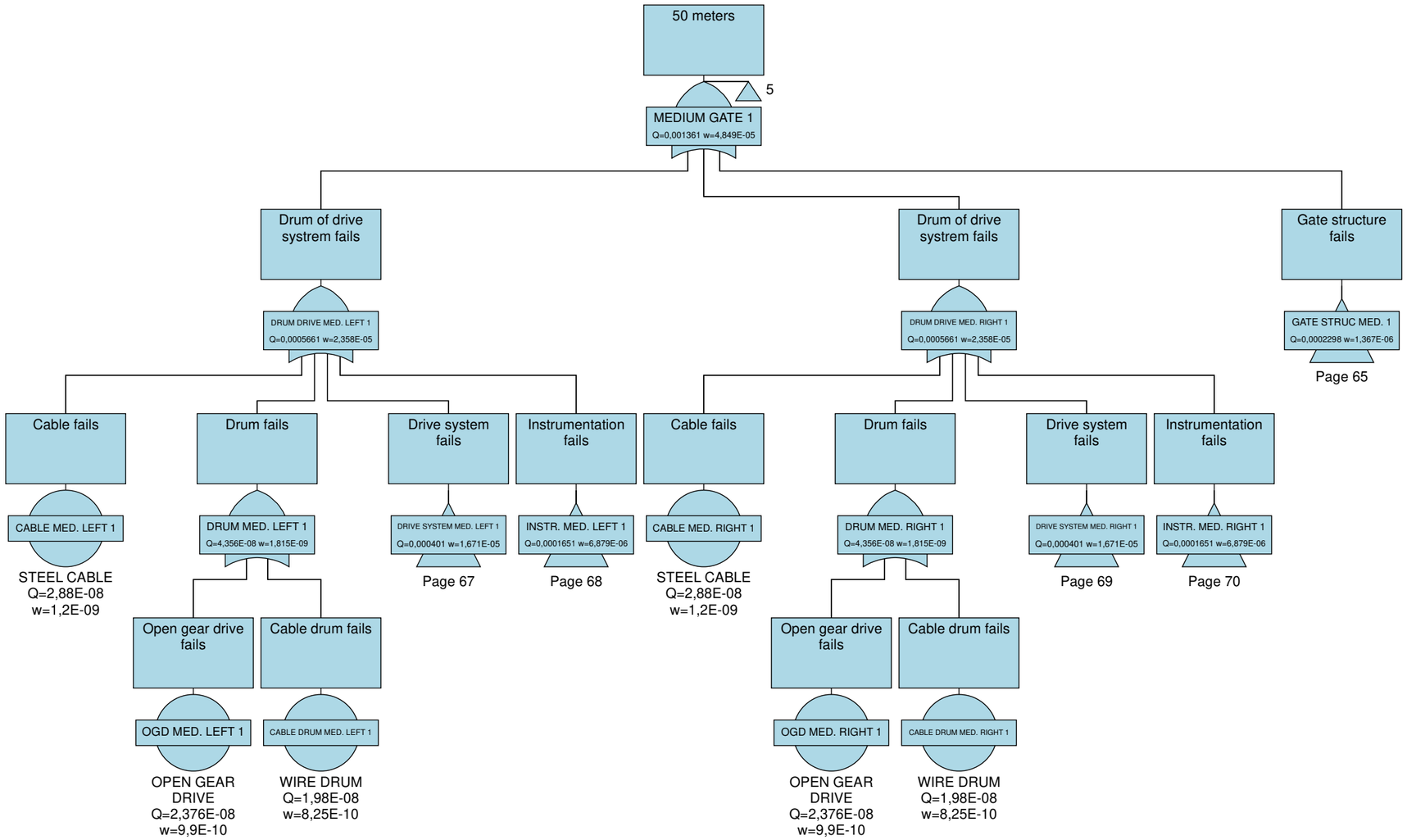

25 meters
4
SMALL GATE 1
Q=0,0007958 w=2,495E-05
Cable fails
Drum fails
Drum fails
Axles fail
Drive system fails
Instrumentation fails
Gate structure fails
CABLE SMALL1
DRUM SMALL LEFT 1
Q=4,356E-08 w=1,815E-09
DRUM SMALL RIGHT 1
Q=4,356E-08 w=1,815E-09
AXLES SMALL 1
Q=3,168E-08 w=1,32E-09
DRIVE SYSTEM SMALL 1
Q=0,000401 w=1,671E-05
INSTRUMENTATION SMALL 1
Q=0,0001651 w=6,879E-06
GATE STRUC SMALL 1
Q=0,0002298 w=1,367E-06
STEEL CABLE
Q=2,88E-08
w=1,2E-09
Page 61
Page 62
Page 63
Page 64
Open gear drive fails
Cable drum fails
Open gear drive fails
Cable drum fails
OGD SMALL LEFT 1
CABLE DRUM SMALL LEFT 1
OGD SMALL RIGHT 1
CABLE DRUM SMALL RIGHT 1
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10

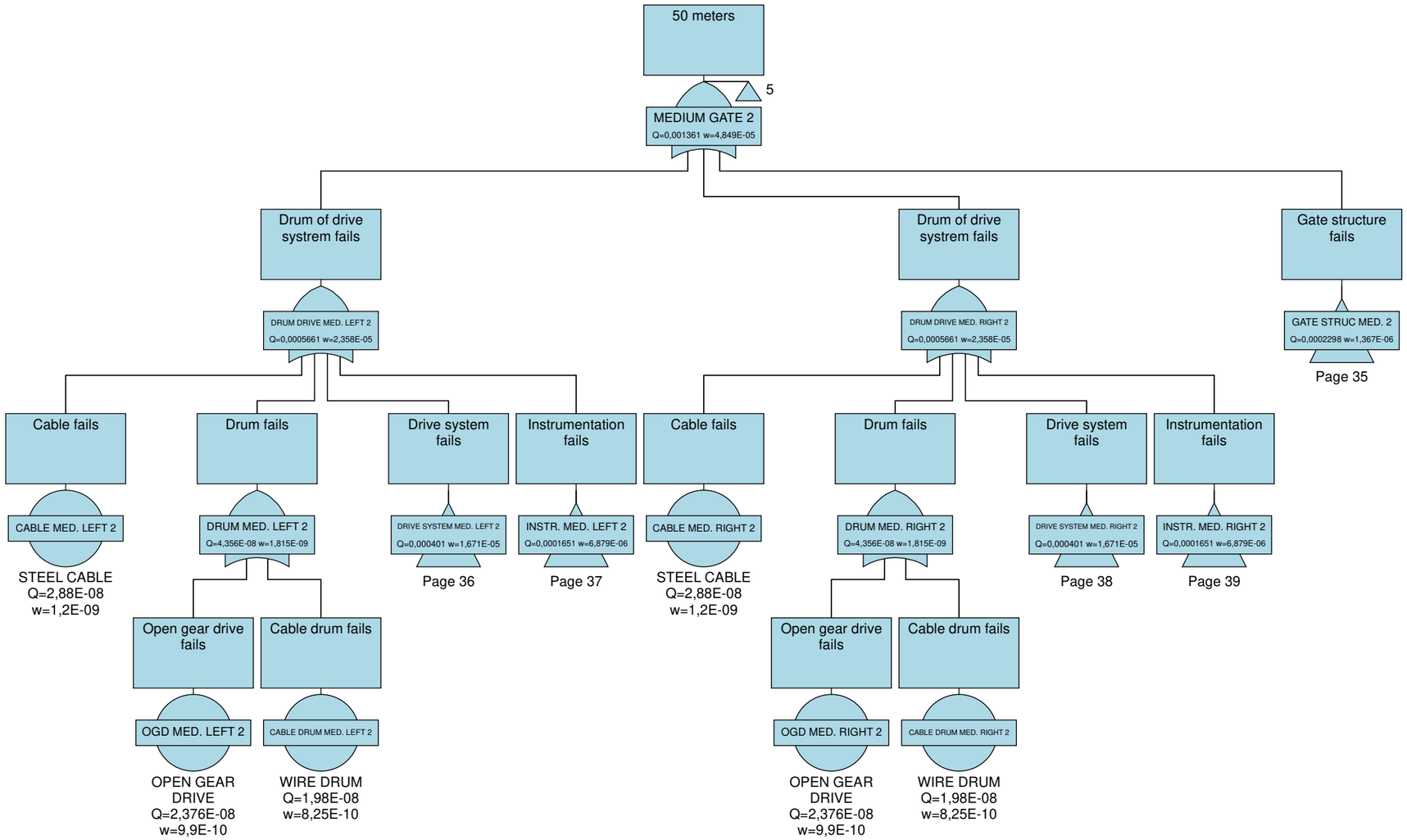

25 meters
4
SMALL GATE 2
Q=0,0007958 w=2,495E-05
Cable fails
Drum fails
Drum fails
Axles fail
Drive system fails
Instrumentation fails
Gate structure fails
CABLE SMALL2
STEEL CABLE
Q=2,88E-08
w=1,2E-09
DRUM SMALL LEFT 2
Q=4,356E-08 w=1,815E-09
DRUM SMALL RIGHT 2
Q=4,356E-08 w=1,815E-09
AXLES SMALL 2
Q=3,168E-08 w=1,32E-09
DRIVE SYSTEM SMALL 2
Q=0,000401 w=1,671E-05
INSTRUMENTATION SMALL 2
Q=0,0001651 w=6,879E-06
GATE STRUC SMALL 2
Q=0,0002298 w=1,367E-06
Page 23
Page 24
Page 25
Page 26
Open gear drive fails
Cable drum fails
Open gear drive fails
Cable drum fails
OGD SMALL LEFT 2
CABLE DRUM SMALL LEFT 2
OGD SMALL RIGHT 2
CABLE DRUM SMALL RIGHT 2
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10

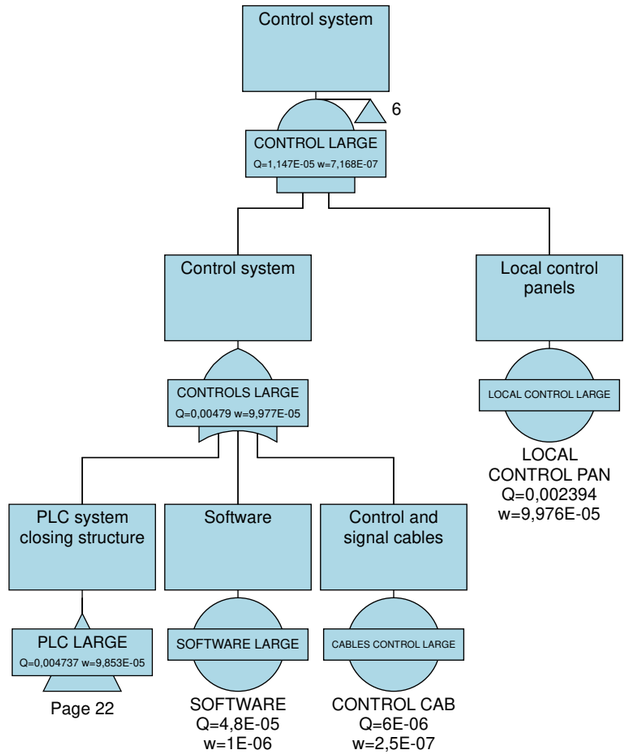

25 meters
4
SMALL GATE 3
Q=0,0007958 w=2,495E-05
Cable fails
Drum fails
Drum fails
Axles fail
Drive system fails
Instrumentation fails
Gate structure fails
CABLE SMALL3
STEEL CABLE
Q=2,88E-08
w=1,2E-09
DRUM SMALL LEFT 3
Q=4,356E-08 w=1,815E-09
DRUM SMALL RIGHT 3
Q=4,356E-08 w=1,815E-09
AXLES SMALL 3
Q=3,168E-08 w=1,32E-09
DRIVE SYSTEM SMALL 3
Q=0,000401 w=1,671E-05
INSTRUMENTATION SMALL 3
Q=0,0001651 w=6,879E-06
GATE STRUC SMALL 3
Q=0,0002298 w=1,367E-06
Page 27
Page 28
Page 29
Page 30
Open gear drive fails
Cable drum fails
Open gear drive fails
Cable drum fails
OGD SMALL LEFT 3
CABLE DRUM SMALL LEFT 3
OGD SMALL RIGHT 3
CABLE DRUM SMALL RIGHT 3
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10

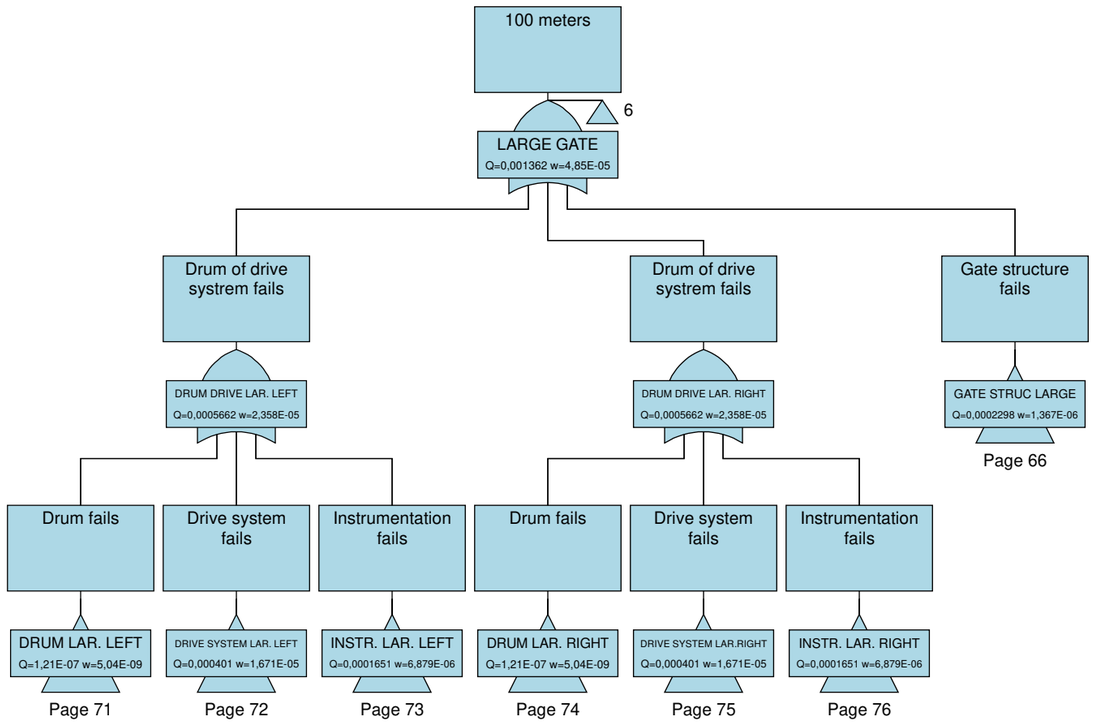

25 meters
4
SMALL GATE 4
Q=0,0007958 w=2,495E-05
Cable fails
Drum fails
Drum fails
Axles fail
Drive system fails
Instrumentation fails
Gate structure fails
CABLE SMALL4
STEEL CABLE
Q=2,88E-08
w=1,2E-09
DRUM SMALL LEFT 4
Q=4,356E-08 w=1,815E-09
DRUM SMALL RIGHT 4
Q=4,356E-08 w=1,815E-09
AXLES SMALL 4
Q=3,168E-08 w=1,32E-09
DRIVE SYSTEM SMALL 4
Q=0,000401 w=1,671E-05
INSTRUMENTATION SMALL 4
Q=0,0001651 w=6,879E-06
GATE STRUC SMALL 4
Q=0,0002298 w=1,367E-06
Page 31
Page 32
Page 33
Page 34
Open gear drive fails
Cable drum fails
Open gear drive fails
Cable drum fails
OGD SMALL LEFT 4
CABLE DRUM SMALL LEFT 4
OGD SMALL RIGHT 4
CABLE DRUM SMALL RIGHT 4
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10

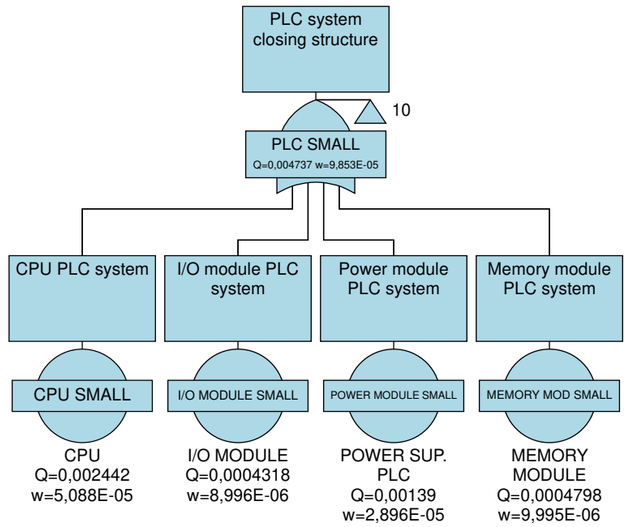

Control system
5
CONTROL MEDIUM
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
CONTROLS MEDIUM
Q=0,00479 w=9,977E-05
LOCAL CONTROL MEDIUM
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC MEDIUM
Q=0,004737 w=9,853E-05
SOFTWARE MEDIUM
CABLES CONTROL MEDIUM
Page 21
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

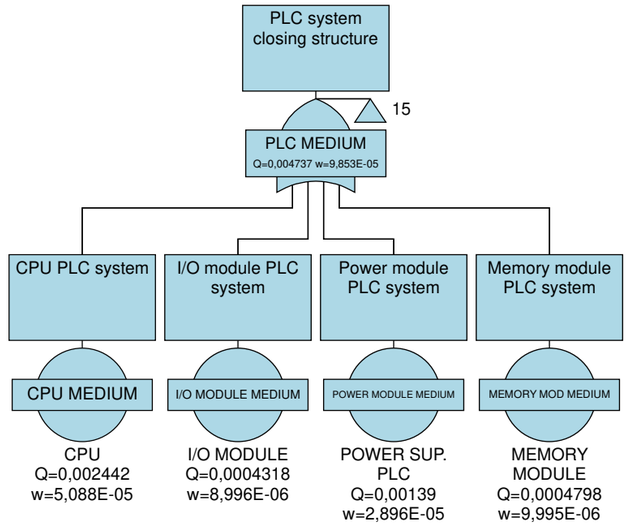

50 meters
5
MEDIUM GATE 1
Q=0,001361 w=4,849E-05
Drum of drive systrem fails
DRUM DRIVE MED. LEFT 1
Q=0,0005661 w=2,358E-05
Cable fails
CABLE MED. LEFT 1
STEEL CABLE
Q=2,88E-08
w=1,2E-09
Drum fails
DRUM MED. LEFT 1
Q=4,356E-08 w=1,815E-09
Open gear drive fails
OGD MED. LEFT 1
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
Cable drum fails
CABLE DRUM MED. LEFT 1
WIRE DRUM
Q=1,98E-08
w=8,25E-10
Drive system fails
DRIVE SYSTEM MED. LEFT 1
Q=0,000401 w=1,671E-05
Page 67
Instrumentation fails
INSTR. MED. LEFT 1
Q=0,0001651 w=6,879E-06
Page 68
Drum of drive systrem fails
DRUM DRIVE MED. RIGHT 1
Q=0,0005661 w=2,358E-05
Cable fails
CABLE MED. RIGHT 1
STEEL CABLE
Q=2,88E-08
w=1,2E-09
Drum fails
DRUM MED. RIGHT 1
Q=4,356E-08 w=1,815E-09
Open gear drive fails
OGD MED. RIGHT 1
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
Cable drum fails
CABLE DRUM MED. RIGHT 1
WIRE DRUM
Q=1,98E-08
w=8,25E-10
Drive system fails
DRIVE SYSTEM MED. RIGHT 1
Q=0,000401 w=1,671E-05
Page 69
Instrumentation fails
INSTR. MED. RIGHT 1
Q=0,0001651 w=6,879E-06
Page 70
Gate structure fails
GATE STRUC MED. 1
Q=0,0002298 w=1,367E-06
Page 65

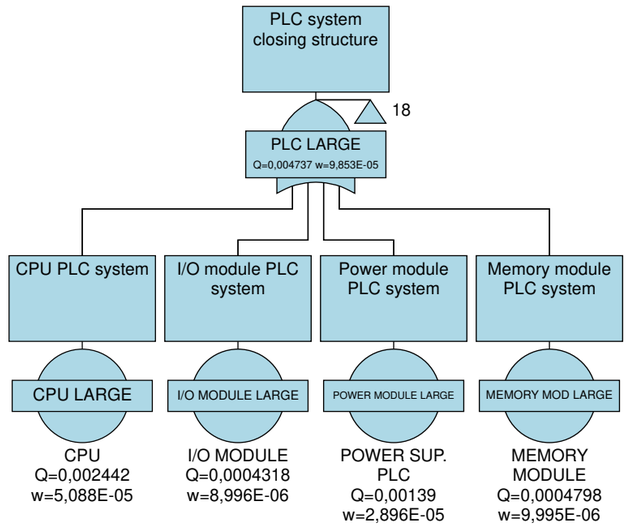

50 meters
5
MEDIUM GATE 2
Q=0,001361 w=4,849E-05
Drum of drive systrem fails
DRUM DRIVE MED. LEFT 2
Q=0,0005661 w=2,358E-05
Drum of drive systrem fails
DRUM DRIVE MED. RIGHT 2
Q=0,0005661 w=2,358E-05
Gate structure fails
GATE STRUC MED. 2
Q=0,0002298 w=1,367E-06
Page 35
Cable fails
CABLE MED. LEFT 2
STEEL CABLE
Q=2,88E-08
w=1,2E-09
Drum fails
DRUM MED. LEFT 2
Q=4,356E-08 w=1,815E-09
Drive system fails
DRIVE SYSTEM MED. LEFT 2
Q=0,000401 w=1,671E-05
Page 36
Instrumentation fails
INSTR. MED. LEFT 2
Q=0,0001651 w=6,879E-06
Page 37
Cable fails
CABLE MED. RIGHT 2
STEEL CABLE
Q=2,88E-08
w=1,2E-09
Drum fails
DRUM MED. RIGHT 2
Q=4,356E-08 w=1,815E-09
Drive system fails
DRIVE SYSTEM MED. RIGHT 2
Q=0,000401 w=1,671E-05
Page 38
Instrumentation fails
INSTR. MED. RIGHT 2
Q=0,0001651 w=6,879E-06
Page 39
Open gear drive fails
OGD MED. LEFT 2
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
Cable drum fails
CABLE DRUM MED. LEFT 2
WIRE DRUM
Q=1,98E-08
w=8,25E-10
Open gear drive fails
OGD MED. RIGHT 2
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10
Cable drum fails
CABLE DRUM MED. RIGHT 2
WIRE DRUM
Q=1,98E-08
w=8,25E-10

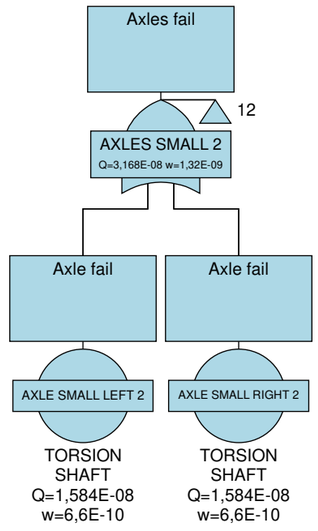

Control system
6
CONTROL LARGE
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
CONTROLS LARGE
Q=0,00479 w=9,977E-05
LOCAL CONTROL LARGE
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC LARGE
Q=0,004737 w=9,853E-05
SOFTWARE LARGE
CABLES CONTROL LARGE
Page 22
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

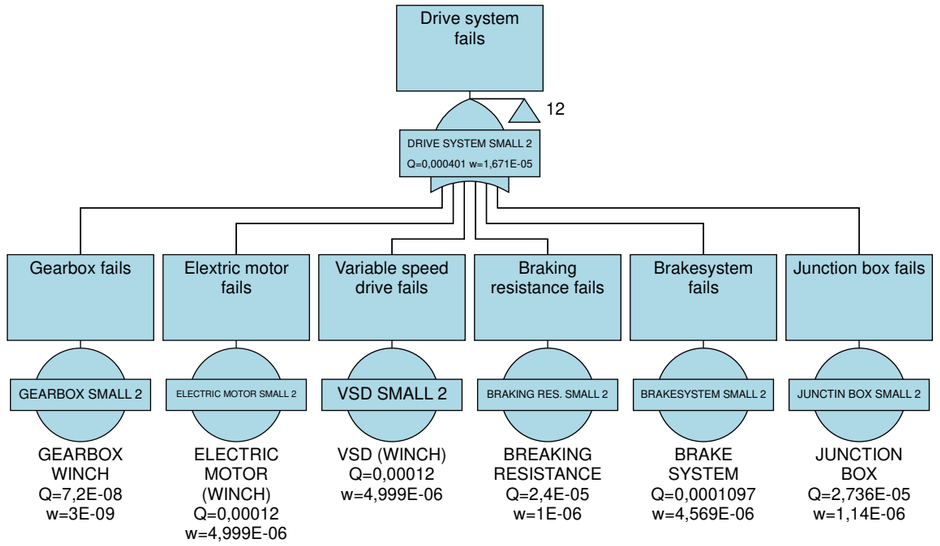

100 meters
6
LARGE GATE
Q=0,001362 w=4,85E-05
Drum of drive systrem fails
Drum of drive systrem fails
Gate structure fails
DRUM DRIVE LAR. LEFT
Q=0,0005662 w=2,358E-05
DRUM DRIVE LAR. RIGHT
Q=0,0005662 w=2,358E-05
GATE STRUC LARGE
Q=0,0002298 w=1,367E-06
Page 66
Drum fails
Drive system fails
Instrumentation fails
Drum fails
Drive system fails
Instrumentation fails
DRUM LAR. LEFT
Q=1,21E-07 w=5,04E-09
DRIVE SYSTEM LAR. LEFT
Q=0,000401 w=1,671E-05
INSTR. LAR. LEFT
Q=0,0001651 w=6,879E-06
DRUM LAR. RIGHT
Q=1,21E-07 w=5,04E-09
DRIVE SYSTEM LAR.RIGHT
Q=0,000401 w=1,671E-05
INSTR. LAR. RIGHT
Q=0,0001651 w=6,879E-06
Page 71
Page 72
Page 73
Page 74
Page 75
Page 76

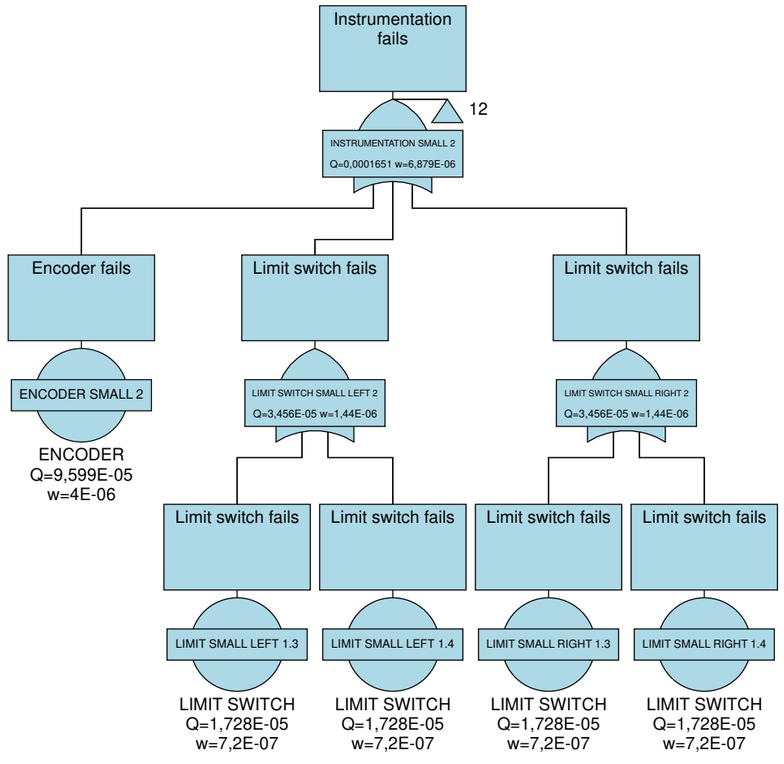

PLC system closing structure
10
PLC SMALL
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU SMALL
I/O MODULE SMALL
POWER MODULE SMALL
MEMORY MOD SMALL
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

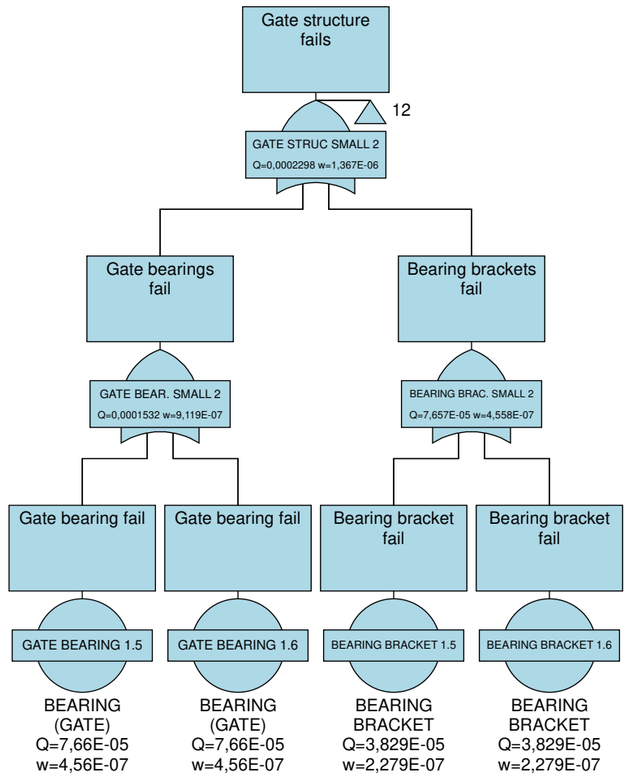

PLC system closing structure
15
PLC MEDIUM
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU MEDIUM
I/O MODULE MEDIUM
POWER MODULE MEDIUM
MEMORY MOD MEDIUM
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

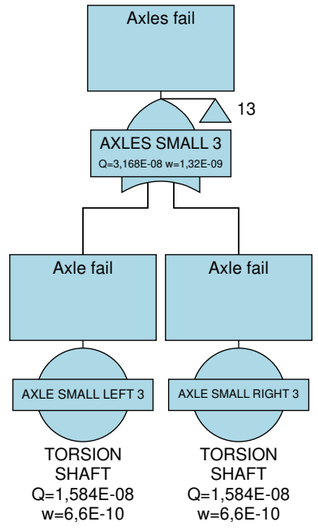

PLC system closing structure
18
PLC LARGE
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU LARGE
I/O MODULE LARGE
POWER MODULE LARGE
MEMORY MOD LARGE
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

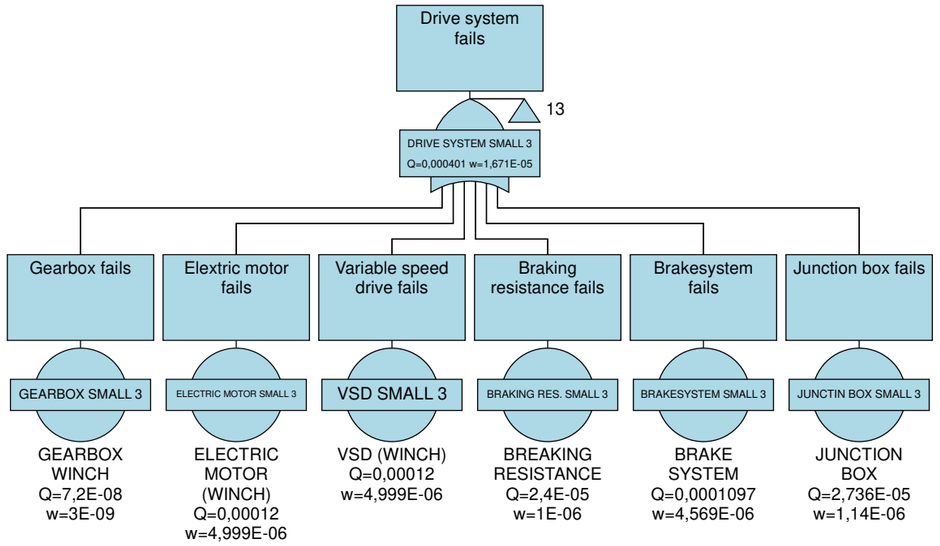

Axles fail
12
AXLES SMALL 2
Q=3,168E-08 w=1,32E-09
Axle fail
Axle fail
AXLE SMALL LEFT 2
AXLE SMALL RIGHT 2
TORSION SHAFT
Q=1,584E-08
w=6,6E-10
TORSION SHAFT
Q=1,584E-08
w=6,6E-10

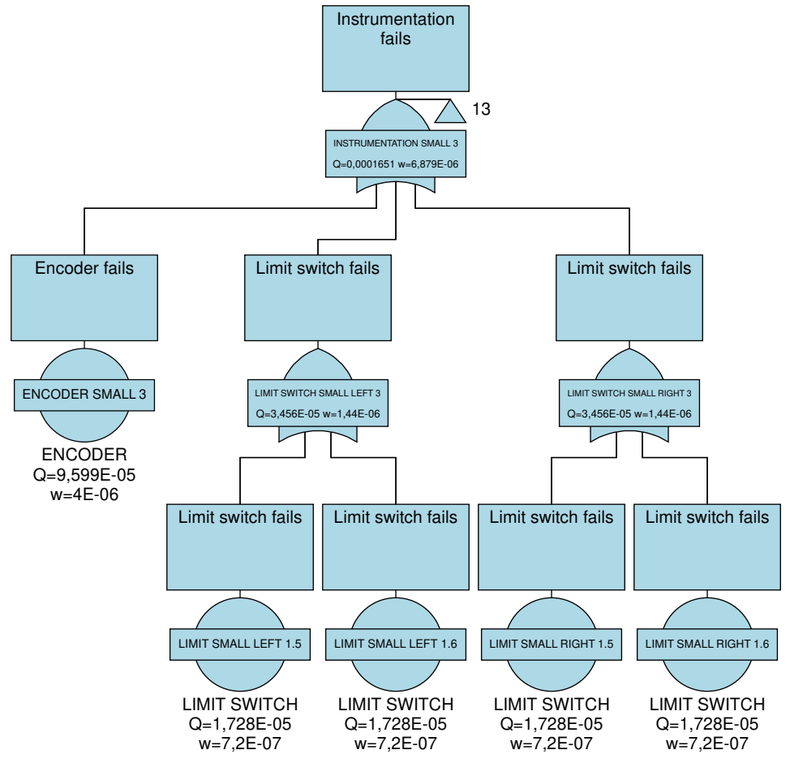

Drive system fails
12
DRIVE SYSTEM SMALL 2
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX SMALL 2
ELECTRIC MOTOR SMALL 2
VSD SMALL 2
BRAKING RES. SMALL 2
BRAKESYSTEM SMALL 2
JUNCTIN BOX SMALL 2
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

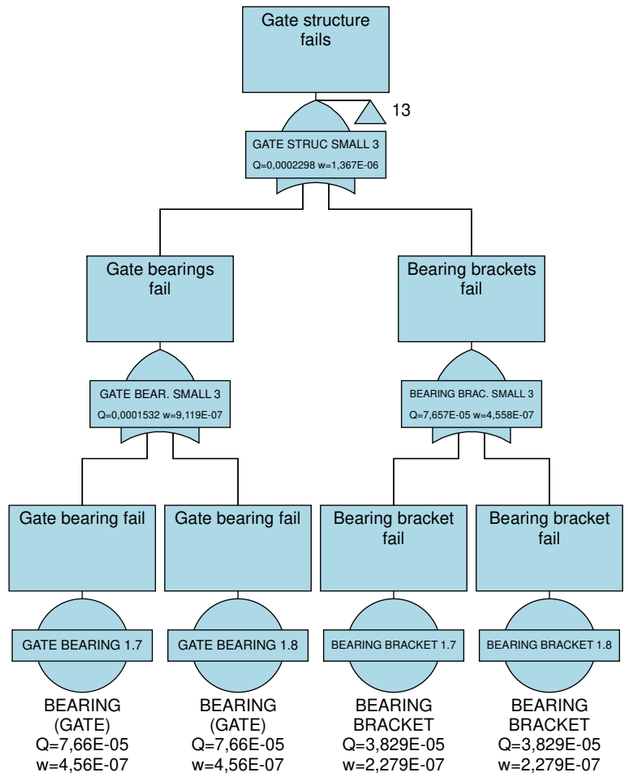

Instrumentation fails
12
INSTRUMENTATION SMALL 2
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER SMALL 2
LIMIT SWITCH SMALL LEFT 2
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH SMALL RIGHT 2
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT SMALL LEFT 1.3
LIMIT SMALL LEFT 1.4
LIMIT SMALL RIGHT 1.3
LIMIT SMALL RIGHT 1.4
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

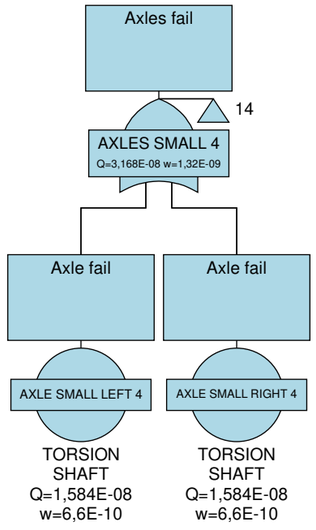

Gate structure fails
12
GATE STRUC SMALL 2
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. SMALL 2
Q=0,0001532 w=9,119E-07
BEARING BRAC. SMALL 2
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.5
GATE BEARING 1.6
BEARING BRACKET 1.5
BEARING BRACKET 1.6
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

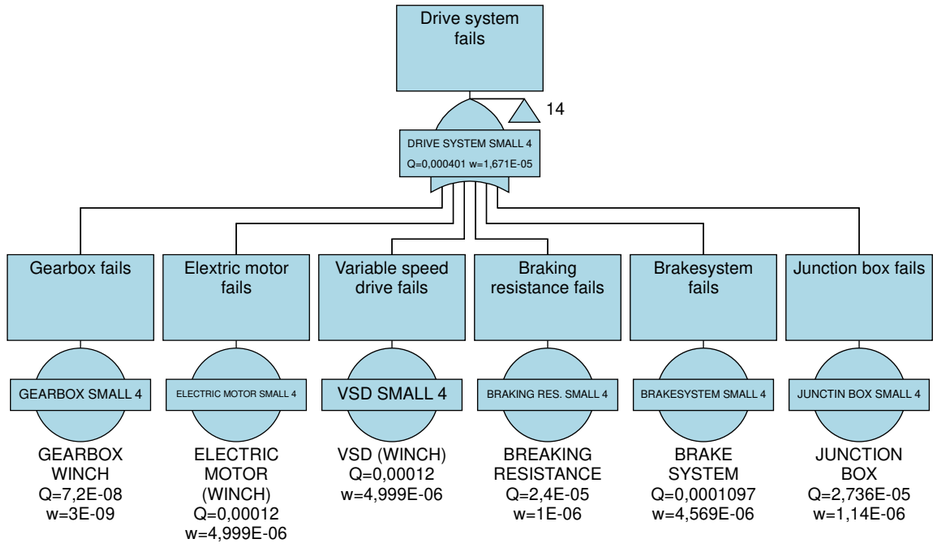

Axles fail
13
AXLES SMALL 3
Q=3,168E-08 w=1,32E-09
Axle fail
Axle fail
AXLE SMALL LEFT 3
AXLE SMALL RIGHT 3
TORSION SHAFT
Q=1,584E-08
w=6,6E-10
TORSION SHAFT
Q=1,584E-08
w=6,6E-10

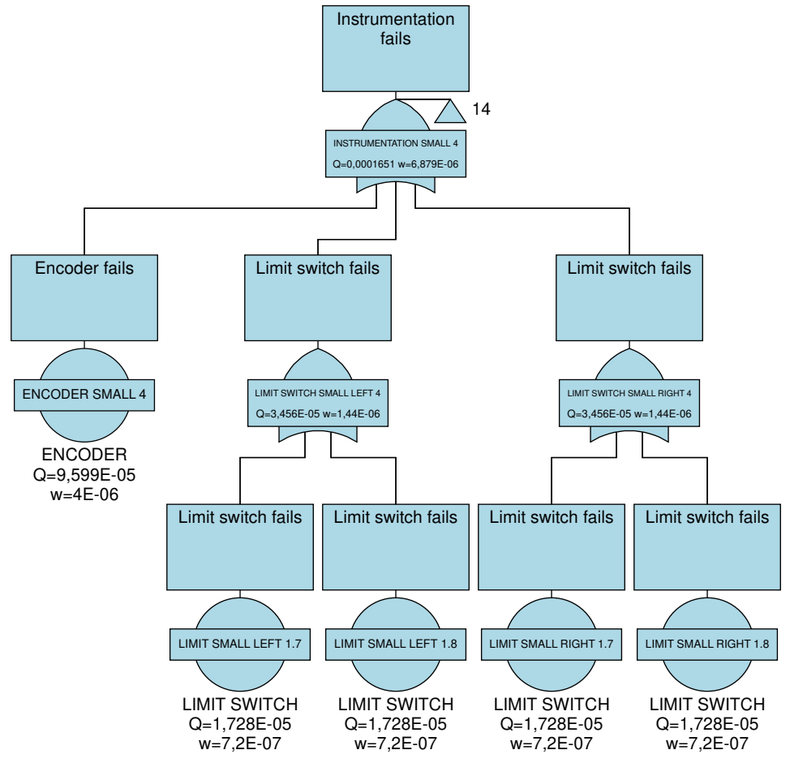

Drive system fails
13
DRIVE SYSTEM SMALL 3
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX SMALL 3
ELECTRIC MOTOR SMALL 3
VSD SMALL 3
BRAKING RES. SMALL 3
BRAKESYSTEM SMALL 3
JUNCTIN BOX SMALL 3
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

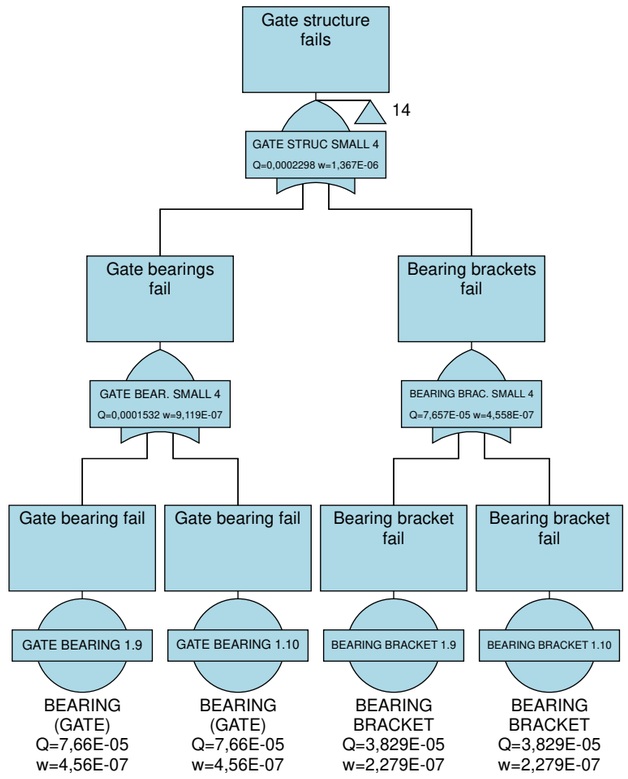

Instrumentation fails
13
INSTRUMENTATION SMALL 3
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER SMALL 3
LIMIT SWITCH SMALL LEFT 3
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH SMALL RIGHT 3
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT SMALL LEFT 1.5
LIMIT SMALL LEFT 1.6
LIMIT SMALL RIGHT 1.5
LIMIT SMALL RIGHT 1.6
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

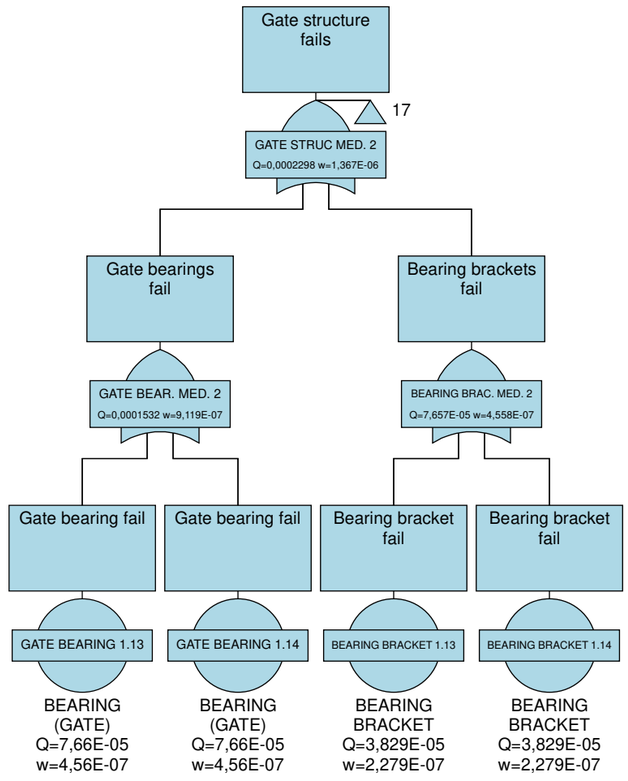

Gate structure fails
13
GATE STRUC SMALL 3
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. SMALL 3
Q=0,0001532 w=9,119E-07
BEARING BRAC. SMALL 3
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.7
GATE BEARING 1.8
BEARING BRACKET 1.7
BEARING BRACKET 1.8
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

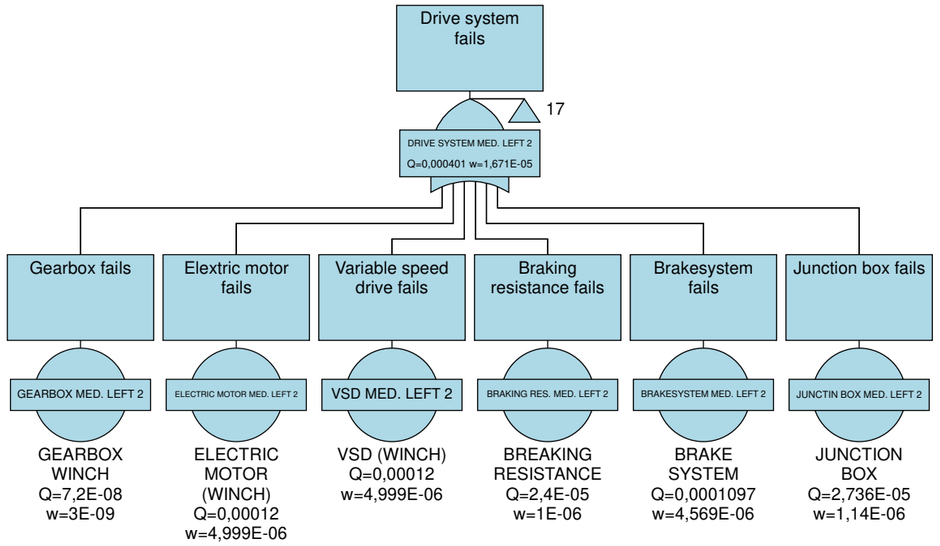

Axles fail
14
AXLES SMALL 4
Q=3,168E-08 w=1,32E-09
Axle fail
Axle fail
AXLE SMALL LEFT 4
AXLE SMALL RIGHT 4
TORSION SHAFT
Q=1,584E-08
w=6,6E-10
TORSION SHAFT
Q=1,584E-08
w=6,6E-10

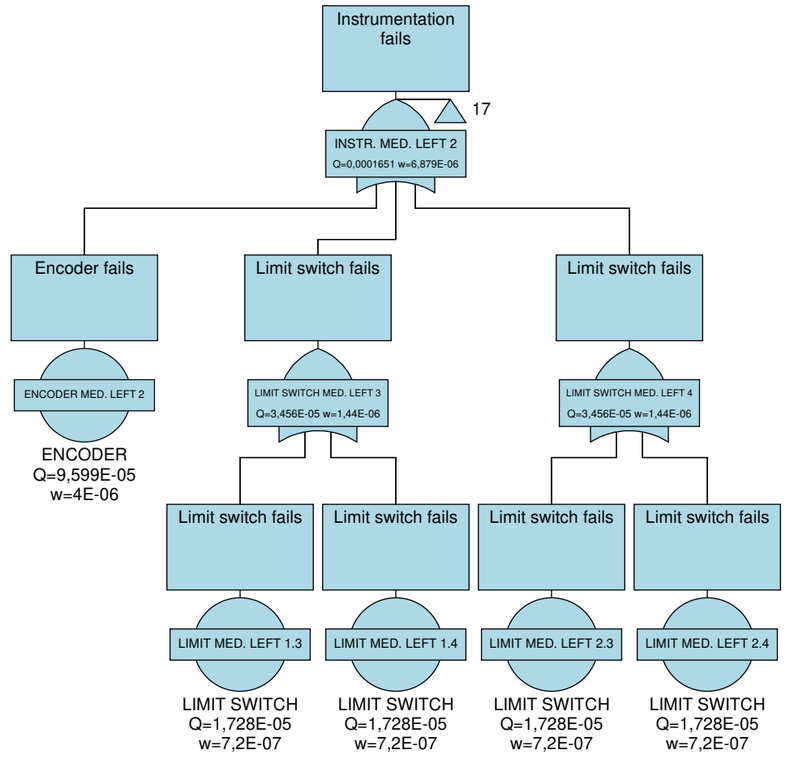

Drive system fails
14
DRIVE SYSTEM SMALL 4
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX SMALL 4
ELECTRIC MOTOR SMALL 4
VSD SMALL 4
BRAKING RES. SMALL 4
BRAKESYSTEM SMALL 4
JUNCTIN BOX SMALL 4
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

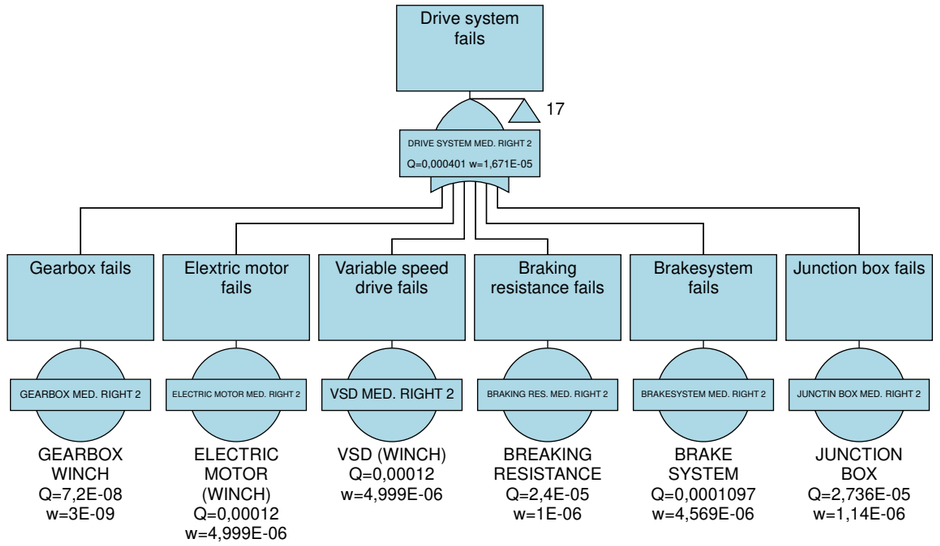

Instrumentation fails
14
INSTRUMENTATION SMALL 4
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER SMALL 4
LIMIT SWITCH SMALL LEFT 4
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH SMALL RIGHT 4
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT SMALL LEFT 1.7
LIMIT SMALL LEFT 1.8
LIMIT SMALL RIGHT 1.7
LIMIT SMALL RIGHT 1.8
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

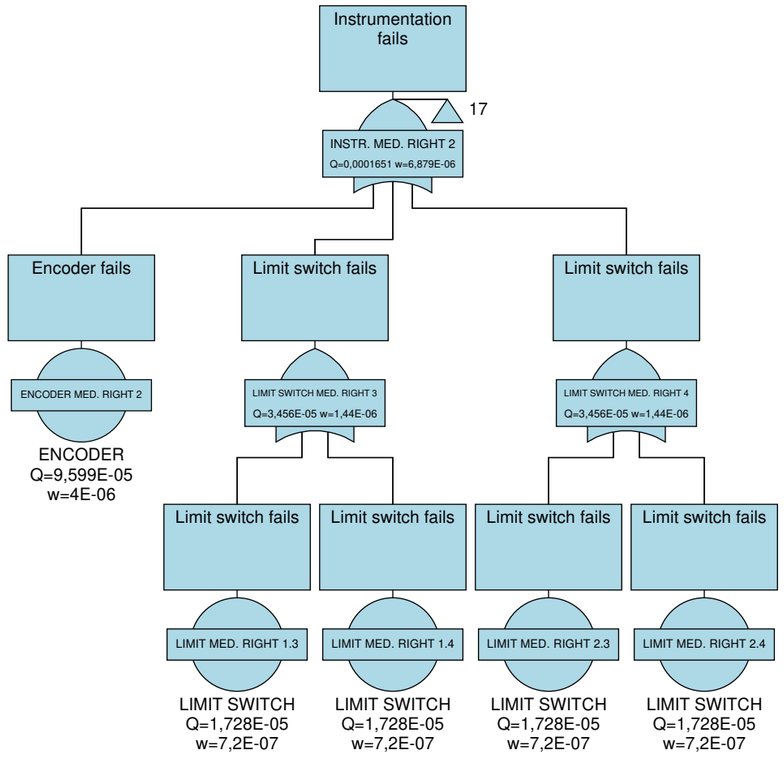

Gate structure fails
14
GATE STRUC SMALL 4
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. SMALL 4
Q=0,0001532 w=9,119E-07
BEARING BRAC. SMALL 4
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.9
GATE BEARING 1.10
BEARING BRACKET 1.9
BEARING BRACKET 1.10
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

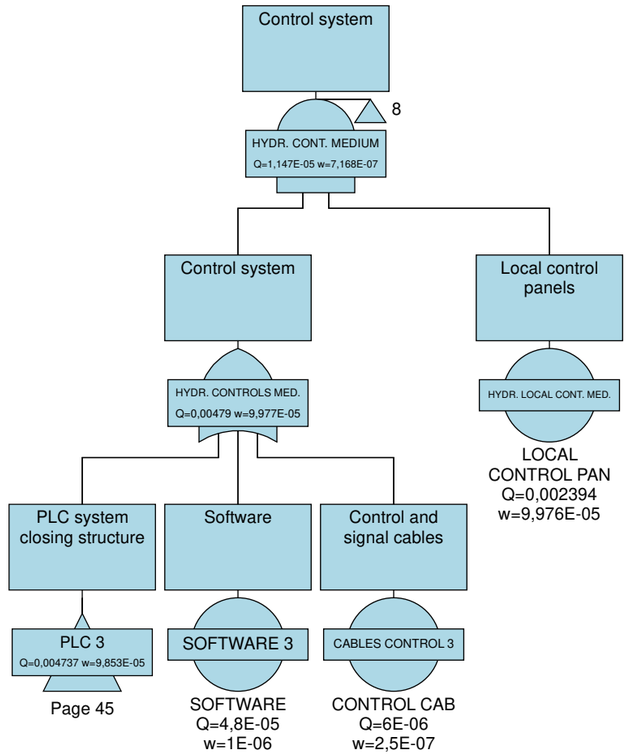

Gate structure fails
17
GATE STRUC MED. 2
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. MED. 2
Q=0,0001532 w=9,119E-07
BEARING BRAC. MED. 2
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.13
GATE BEARING 1.14
BEARING BRACKET 1.13
BEARING BRACKET 1.14
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

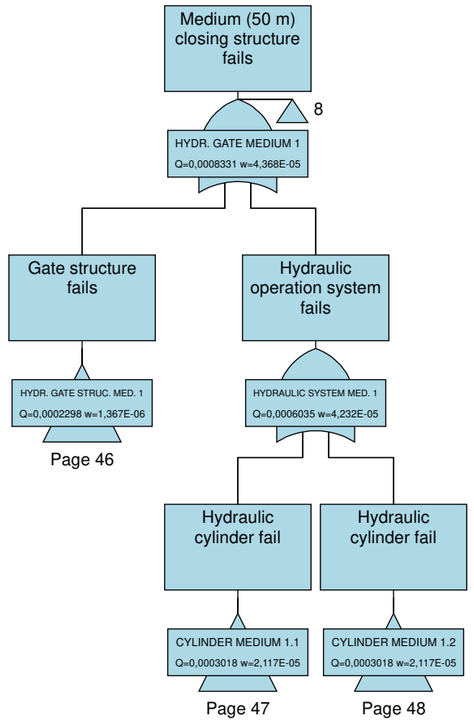

Drive system fails
17
DRIVE SYSTEM MED. LEFT 2
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX MED. LEFT 2
ELECTRIC MOTOR MED. LEFT 2
VSD MED. LEFT 2
BRAKING RES. MED. LEFT 2
BRAKESYSTEM MED. LEFT 2
JUNCTIN BOX MED. LEFT 2
GEARBOX WINCH
Q=7,2E-08
w=3E-09
ELECTRIC MOTOR (WINCH)
Q=0,00012
w=4,999E-06
VSD (WINCH)
Q=0,00012
w=4,999E-06
BREAKING RESISTANCE
Q=2,4E-05
w=1E-06
BRAKE SYSTEM
Q=0,0001097
w=4,569E-06
JUNCTION BOX
Q=2,736E-05
w=1,14E-06

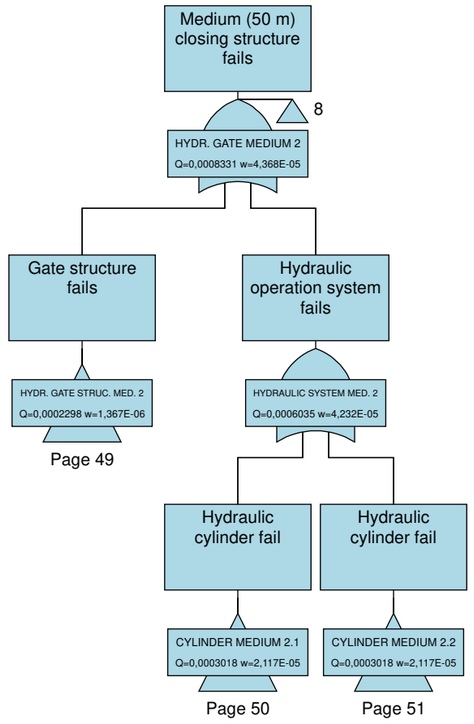

Instrumentation fails
17
INSTR. MED. LEFT 2
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER MED. LEFT 2
LIMIT SWITCH MED. LEFT 3
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH MED. LEFT 4
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT MED. LEFT 1.3
LIMIT MED. LEFT 1.4
LIMIT MED. LEFT 2.3
LIMIT MED. LEFT 2.4
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

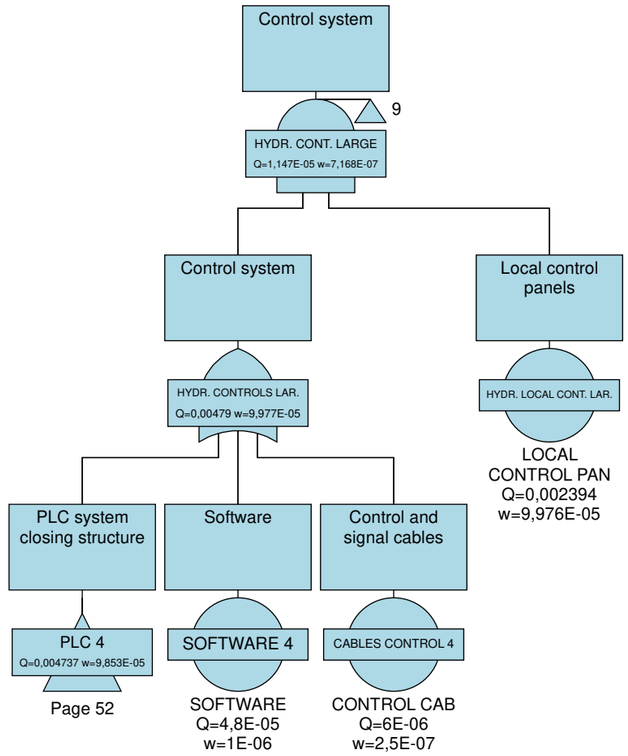

Drive system fails
17
DRIVE SYSTEM MED. RIGHT 2
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX MED. RIGHT 2
ELECTRIC MOTOR MED. RIGHT 2
VSD MED. RIGHT 2
BRAKING RES. MED. RIGHT 2
BRAKESYSTEM MED. RIGHT 2
JUNCTIN BOX MED. RIGHT 2
GEARBOX WINCH
Q=7,2E-08
w=3E-09
ELECTRIC MOTOR (WINCH)
Q=0,00012
w=4,999E-06
VSD (WINCH)
Q=0,00012
w=4,999E-06
BREAKING RESISTANCE
Q=2,4E-05
w=1E-06
BRAKE SYSTEM
Q=0,0001097
w=4,569E-06
JUNCTION BOX
Q=2,736E-05
w=1,14E-06

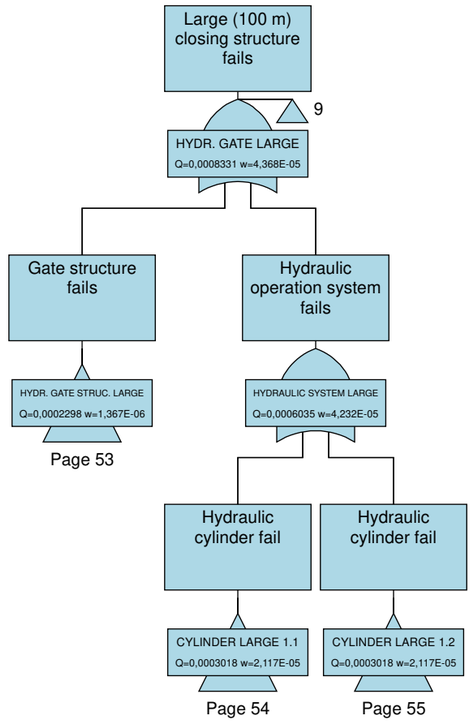

Instrumentation fails
17
INSTR. MED. RIGHT 2
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER MED. RIGHT 2
LIMIT SWITCH MED. RIGHT 3
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH MED. RIGHT 4
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT MED. RIGHT 1.3
LIMIT MED. RIGHT 1.4
LIMIT MED. RIGHT 2.3
LIMIT MED. RIGHT 2.4
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

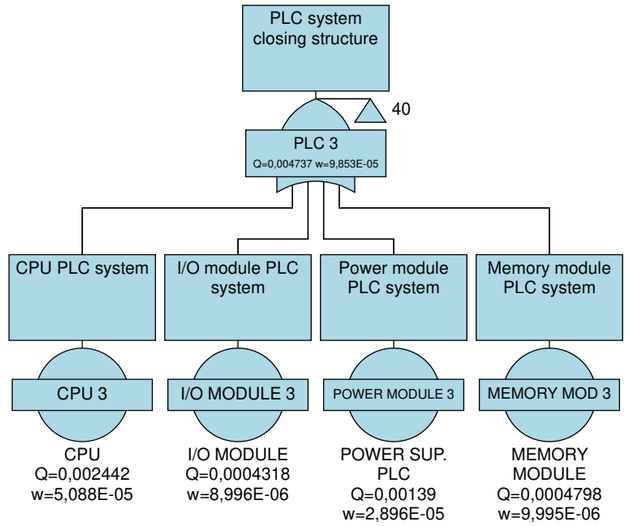

Control system
8
HYDR. CONT. MEDIUM
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
HYDR. CONTROLS MED.
Q=0,00479 w=9,977E-05
HYDR. LOCAL CONT. MED.
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC 3
Q=0,004737 w=9,853E-05
SOFTWARE 3
CABLES CONTROL 3
Page 45
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

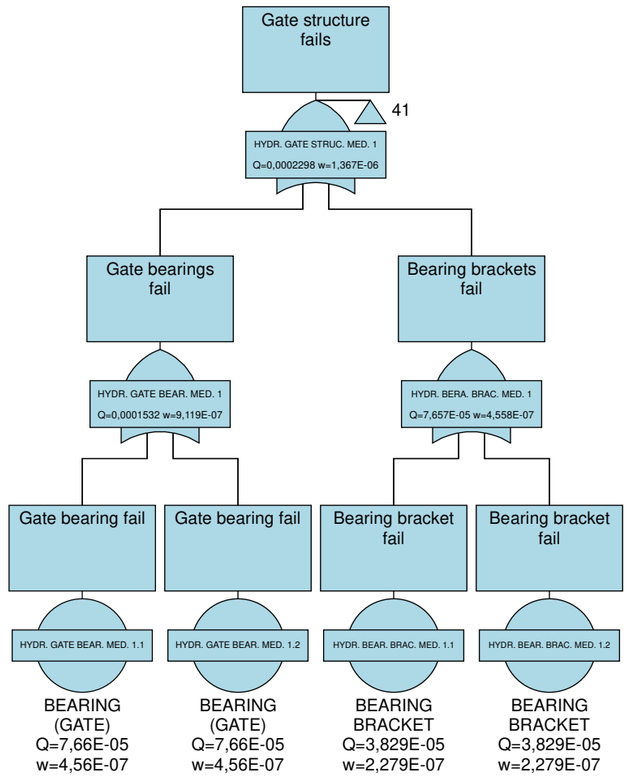

Medium (50 m) closing structure fails
8
HYDR. GATE MEDIUM 1
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. MED. 1
Q=0,0002298 w=1,367E-06
Page 46
HYDRAULIC SYSTEM MED. 1
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER MEDIUM 1.1
Q=0,0003018 w=2,117E-05
CYLINDER MEDIUM 1.2
Q=0,0003018 w=2,117E-05
Page 47
Page 48

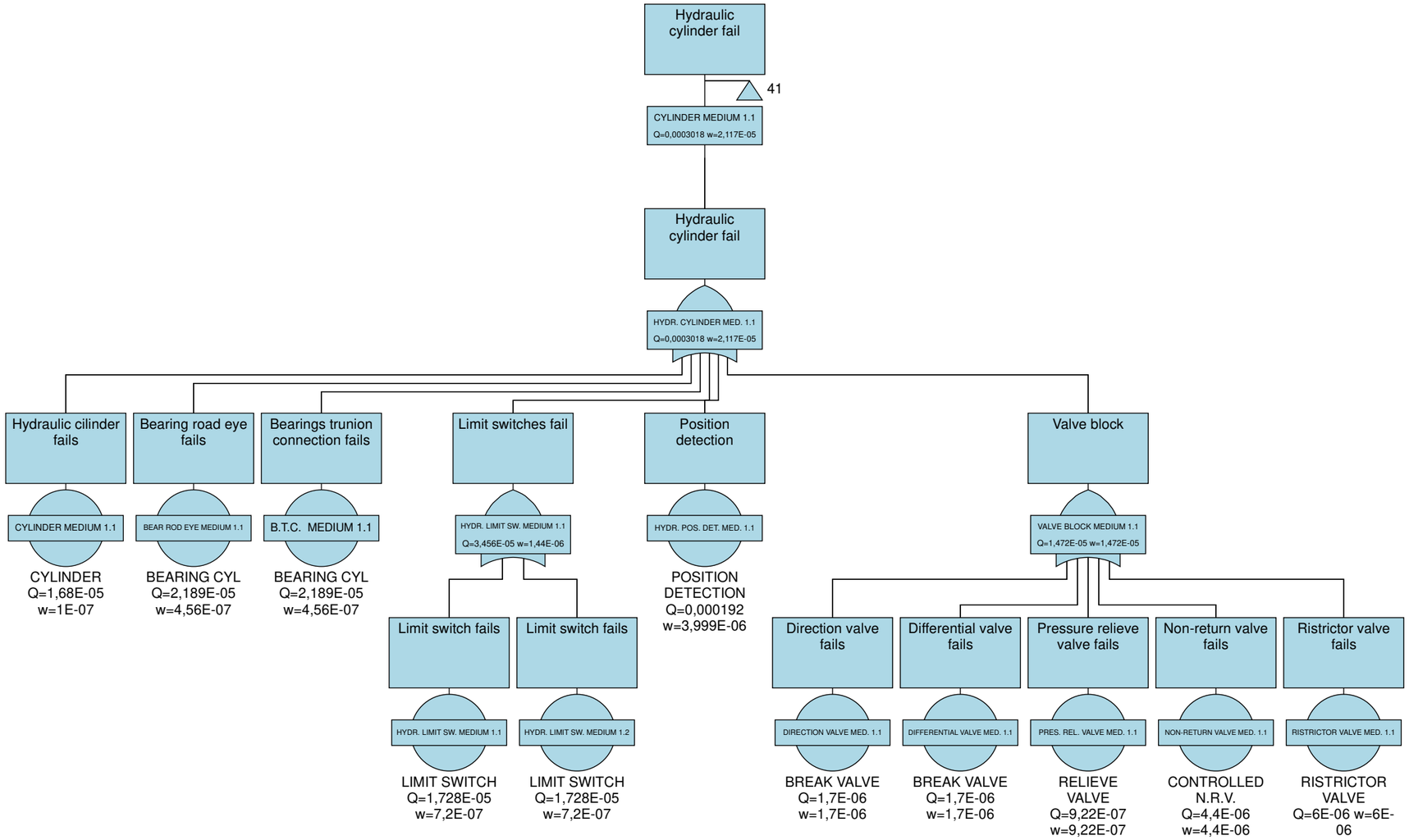

Medium (50 m) closing structure fails
8
HYDR. GATE MEDIUM 2
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. MED. 2
Q=0,0002298 w=1,367E-06
Page 49
HYDRAULIC SYSTEM MED. 2
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER MEDIUM 2.1
Q=0,0003018 w=2,117E-05
CYLINDER MEDIUM 2.2
Q=0,0003018 w=2,117E-05
Page 50
Page 51

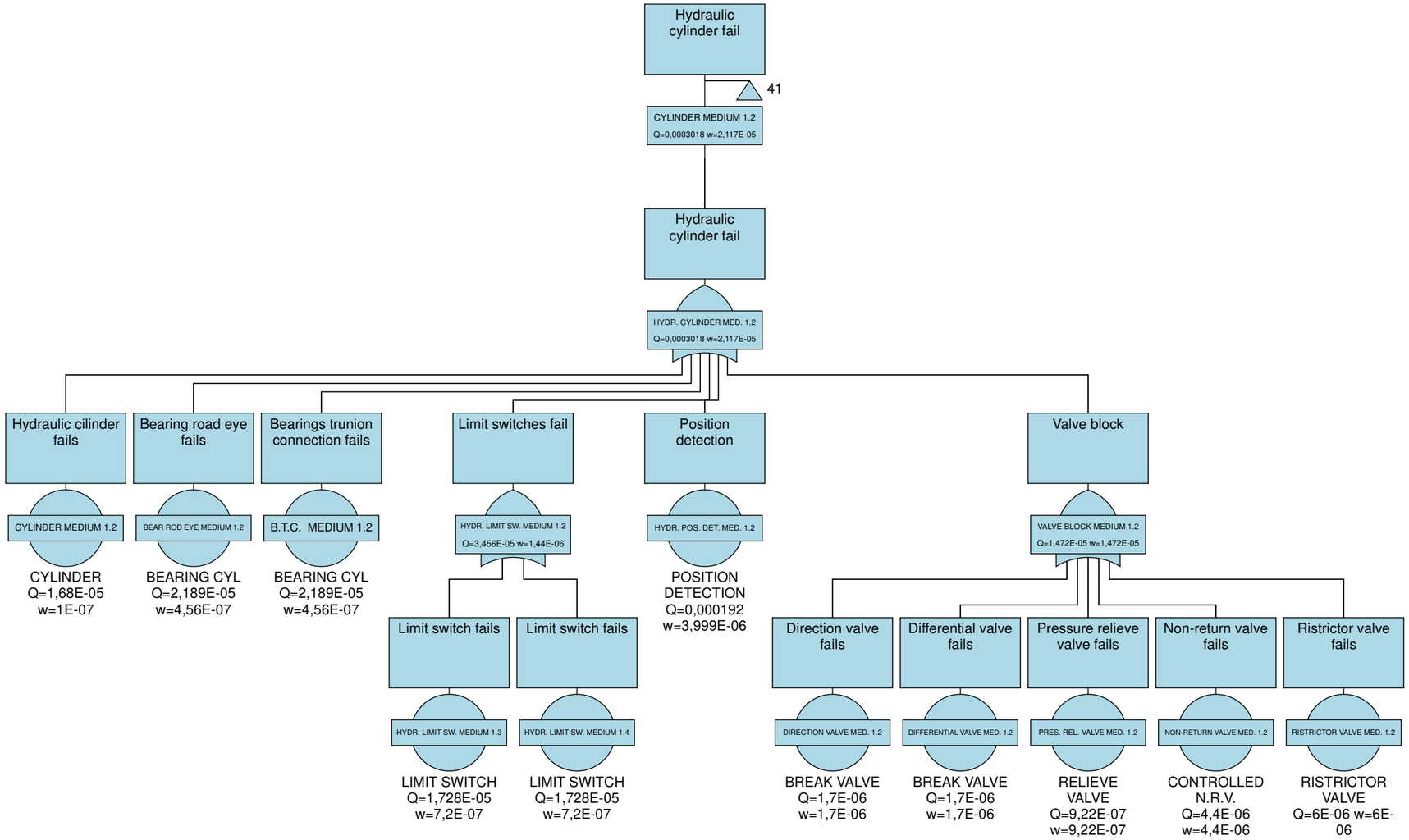

Control system
9
HYDR. CONT. LARGE
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
HYDR. CONTROLS LAR.
Q=0,00479 w=9,977E-05
HYDR. LOCAL CONT. LAR.
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC 4
Q=0,004737 w=9,853E-05
SOFTWARE 4
CABLES CONTROL 4
Page 52
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

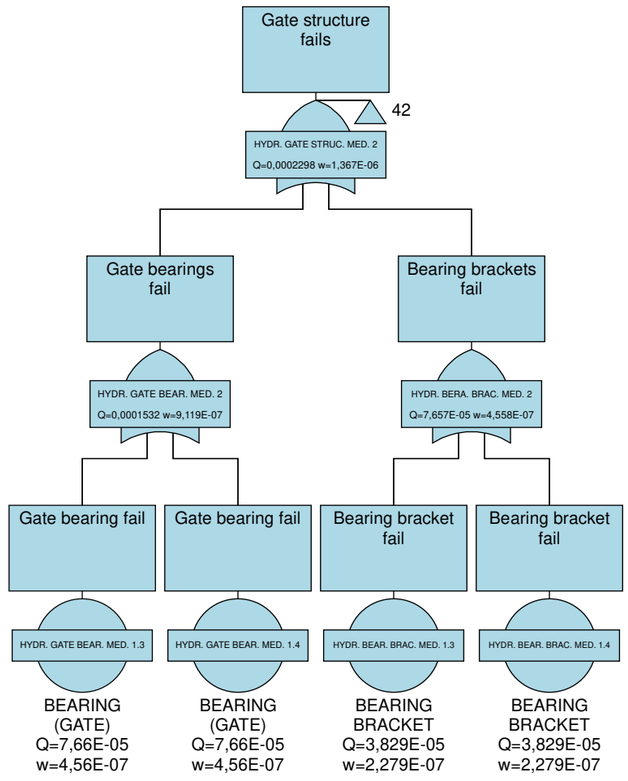

Large (100 m) closing structure fails
9
HYDR. GATE LARGE
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. LARGE
Q=0,0002298 w=1,367E-06
Page 53
HYDRAULIC SYSTEM LARGE
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER LARGE 1.1
Q=0,0003018 w=2,117E-05
CYLINDER LARGE 1.2
Q=0,0003018 w=2,117E-05
Page 54
Page 55

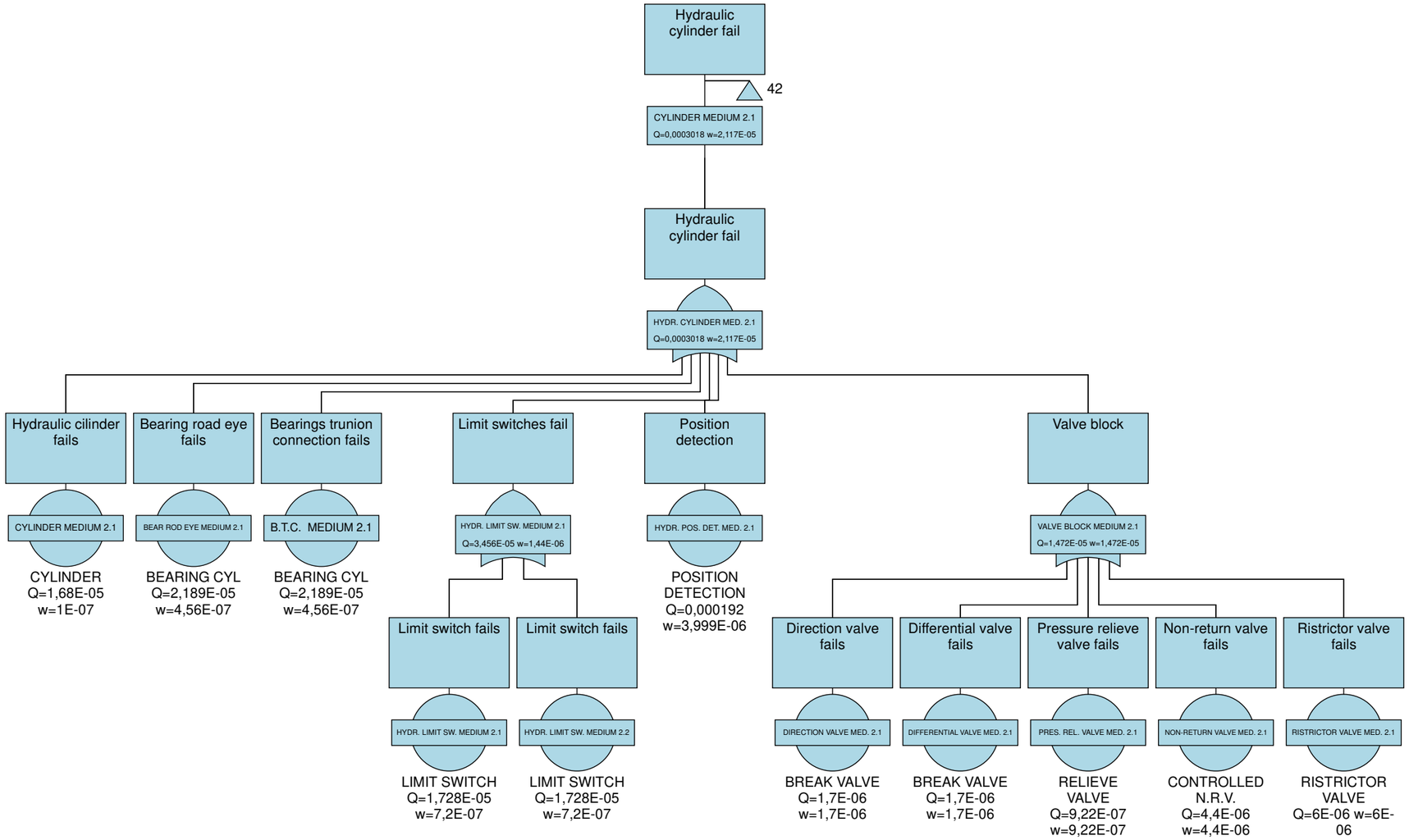

PLC system closing structure
40
PLC 3
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU 3
I/O MODULE 3
POWER MODULE 3
MEMORY MOD 3
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

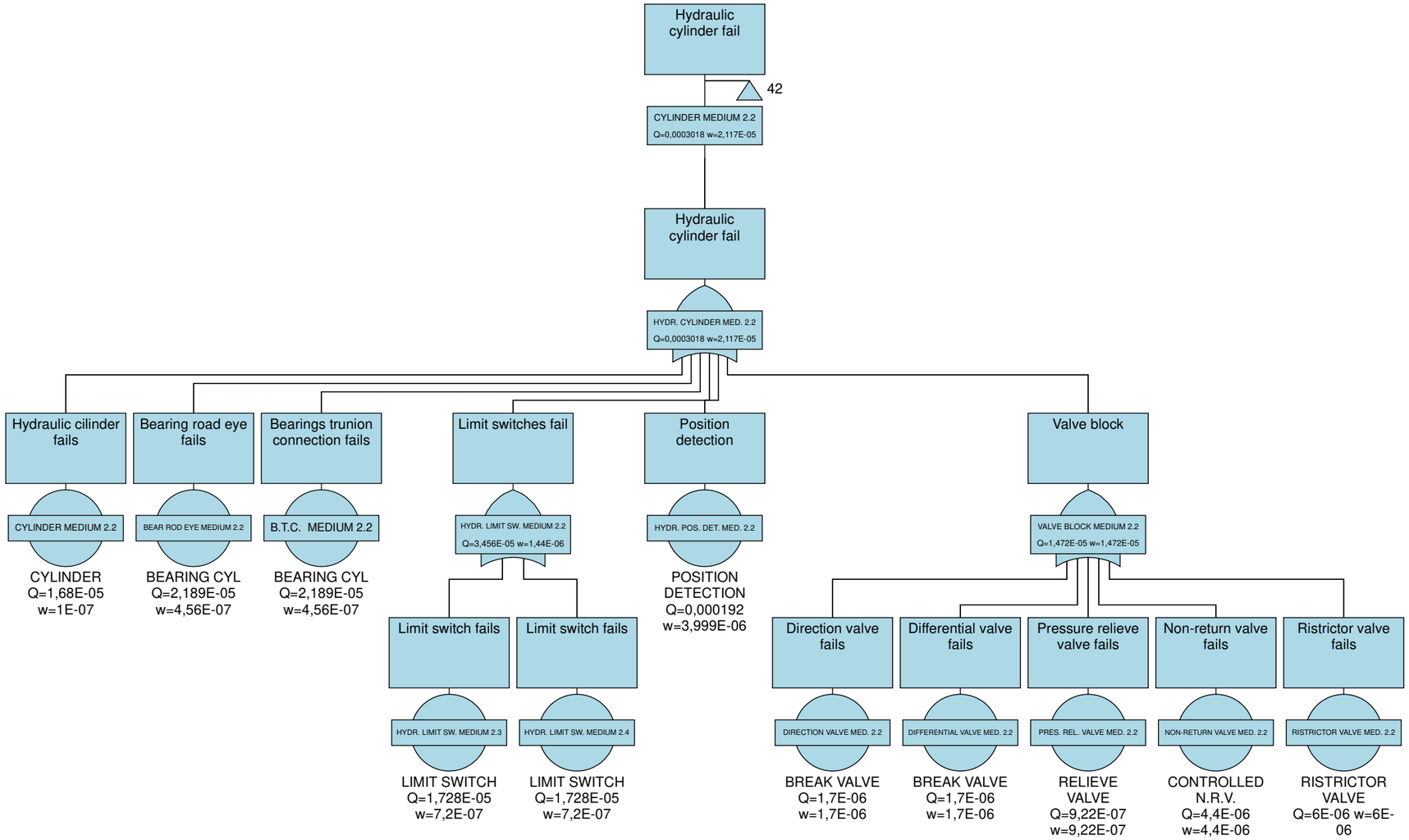

Gate structure fails
41
HYDR. GATE STRUC. MED. 1
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. MED. 1
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. MED. 1
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
HYDR. GATE BEAR. MED. 1.1
HYDR. GATE BEAR. MED. 1.2
HYDR. BEAR. BRAC. MED. 1.1
HYDR. BEAR. BRAC. MED. 1.2
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

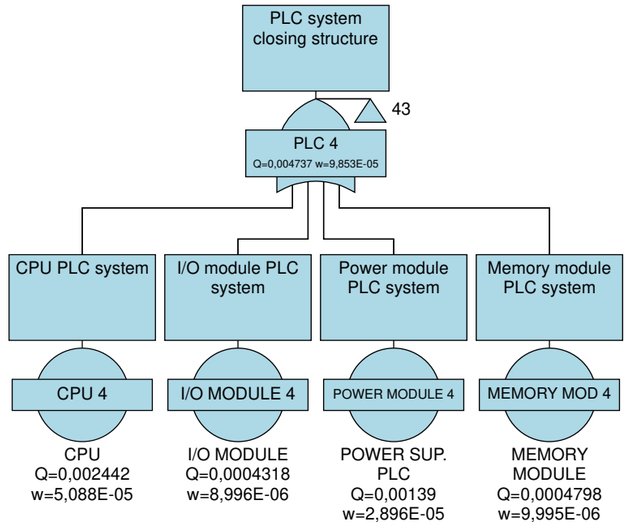

Hydraulic cylinder fail
41
CYLINDER MEDIUM 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER MED. 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER MEDIUM 1.1
BEAR ROD EYE MEDIUM 1.1
B.T.C. MEDIUM 1.1
HYDR. LIMIT SW. MEDIUM 1.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. MED. 1.1
VALVE BLOCK MEDIUM 1.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. MEDIUM 1.1
HYDR. LIMIT SW. MEDIUM 1.2
DIRECTION VALVE MED. 1.1
DIFFERENTIAL VALVE MED. 1.1
PRES. REL. VALVE MED. 1.1
NON-RETURN VALVE MED. 1.1
RISTRICTOR VALVE MED. 1.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

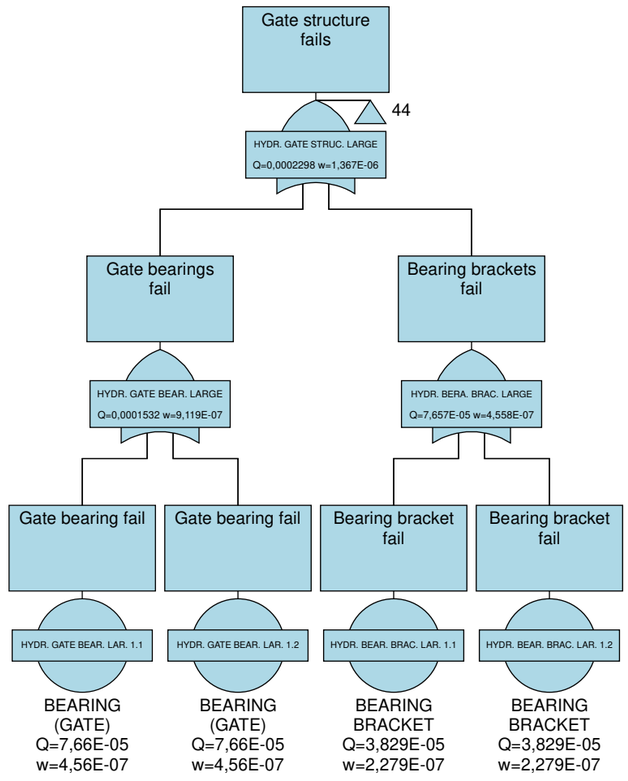

Hydraulic cylinder fail
41
CYLINDER MEDIUM 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER MED. 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER MEDIUM 1.2
BEAR ROD EYE MEDIUM 1.2
B.T.C. MEDIUM 1.2
HYDR. LIMIT SW. MEDIUM 1.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. MED. 1.2
VALVE BLOCK MEDIUM 1.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. MEDIUM 1.3
HYDR. LIMIT SW. MEDIUM 1.4
DIRECTION VALVE MED. 1.2
DIFFERENTIAL VALVE MED. 1.2
PRES. REL. VALVE MED. 1.2
NON-RETURN VALVE MED. 1.2
RISTRICTOR VALVE MED. 1.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

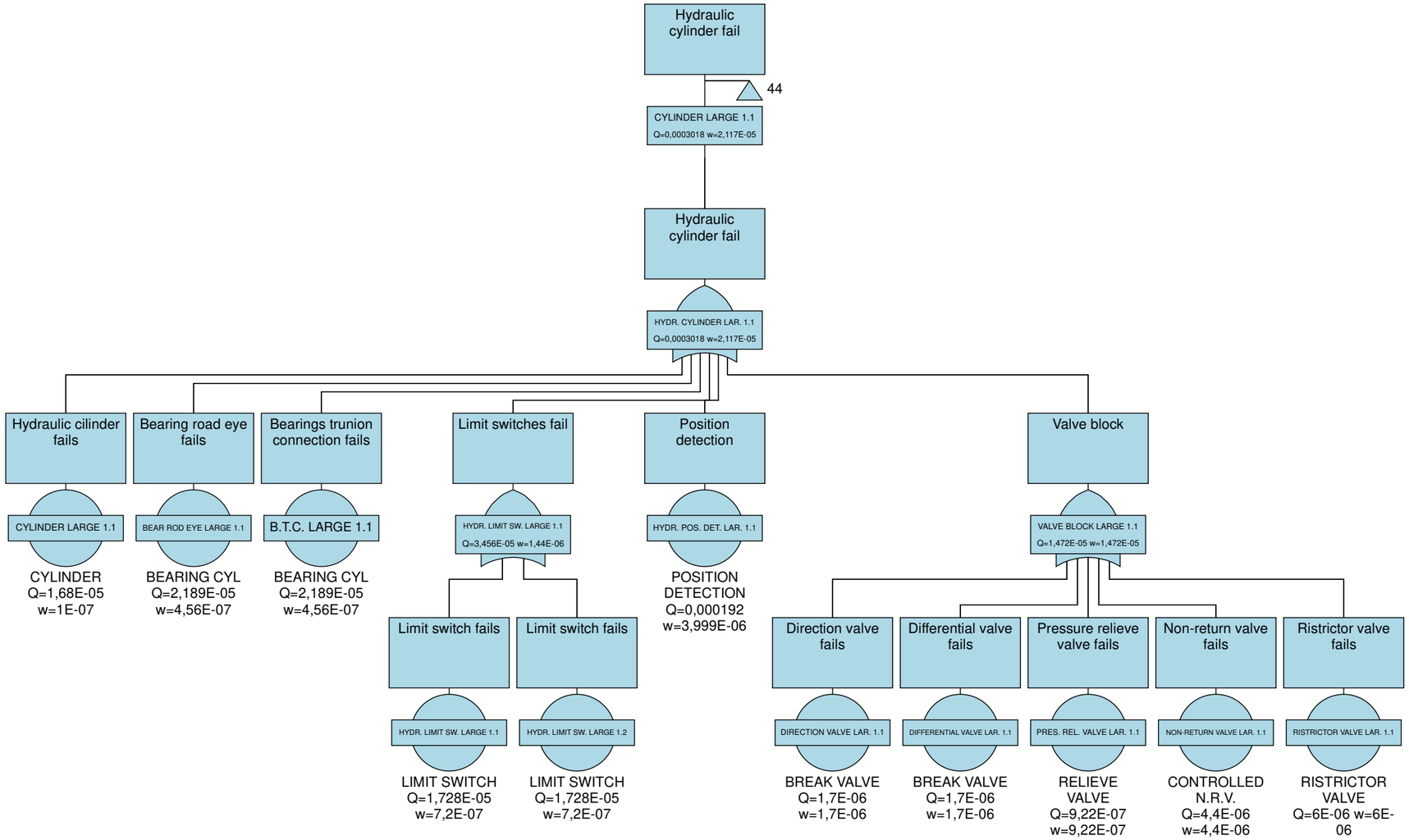

Gate structure fails
42
HYDR. GATE STRUC. MED. 2
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. MED. 2
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. MED. 2
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
HYDR. GATE BEAR. MED. 1.3
HYDR. GATE BEAR. MED. 1.4
HYDR. BEAR. BRAC. MED. 1.3
HYDR. BEAR. BRAC. MED. 1.4
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

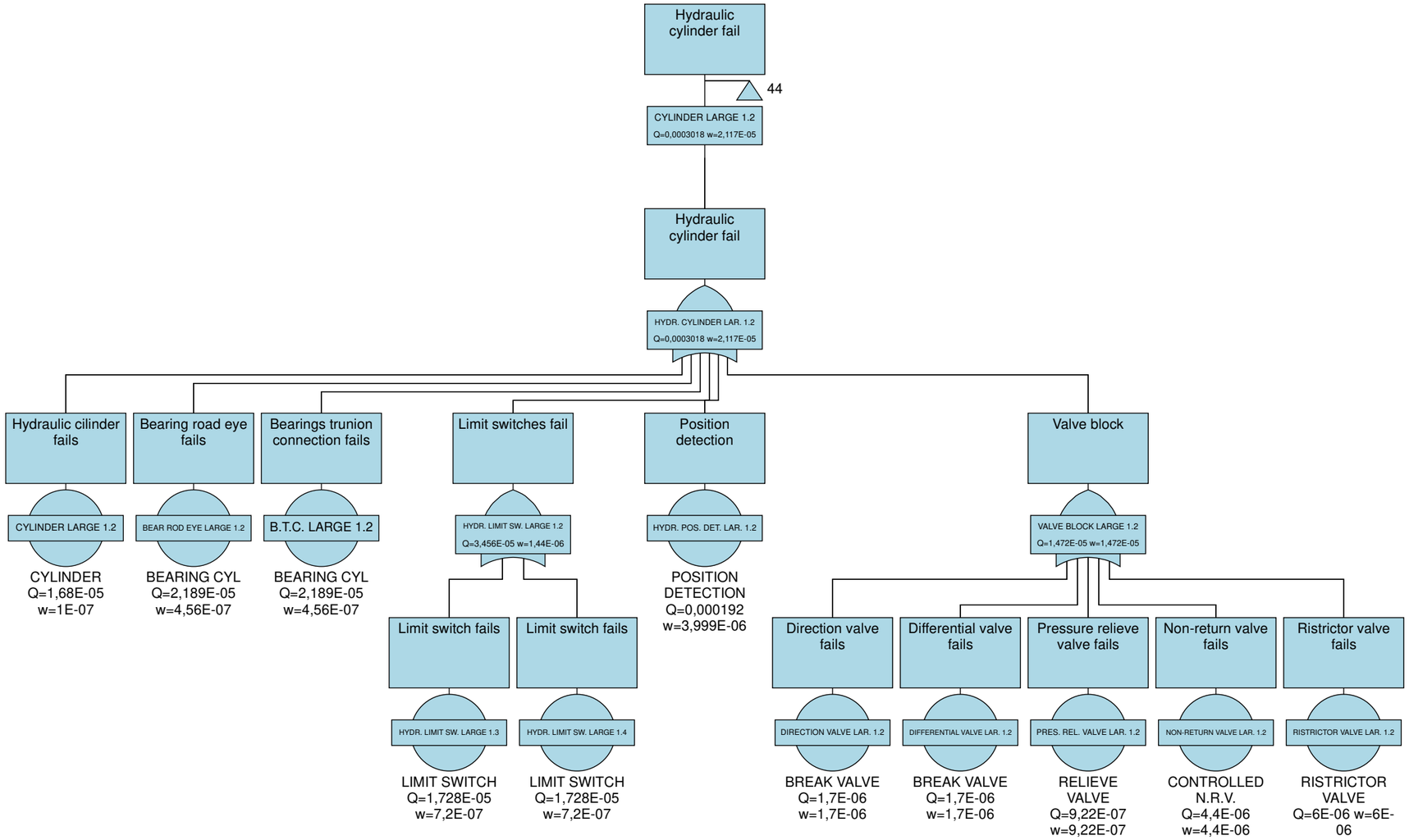

Hydraulic cylinder fail
42
CYLINDER MEDIUM 2.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER MED. 2.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER MEDIUM 2.1
BEAR ROD EYE MEDIUM 2.1
B.T.C. MEDIUM 2.1
HYDR. LIMIT SW. MEDIUM 2.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. MED. 2.1
VALVE BLOCK MEDIUM 2.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. MEDIUM 2.1
HYDR. LIMIT SW. MEDIUM 2.2
DIRECTION VALVE MED. 2.1
DIFFERENTIAL VALVE MED. 2.1
PRES. REL. VALVE MED. 2.1
NON-RETURN VALVE MED. 2.1
RISTRICTOR VALVE MED. 2.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

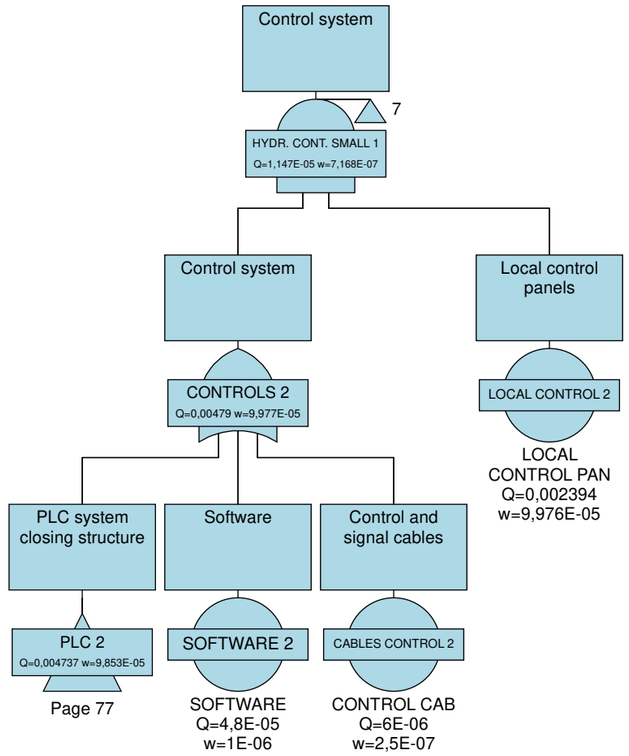

Hydraulic cylinder fail
42
CYLINDER MEDIUM 2.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER MED. 2.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER MEDIUM 2.2
BEAR ROD EYE MEDIUM 2.2
B.T.C. MEDIUM 2.2
HYDR. LIMIT SW. MEDIUM 2.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. MED. 2.2
VALVE BLOCK MEDIUM 2.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. MEDIUM 2.3
HYDR. LIMIT SW. MEDIUM 2.4
DIRECTION VALVE MED. 2.2
DIFFERENTIAL VALVE MED. 2.2
PRES. REL. VALVE MED. 2.2
NON-RETURN VALVE MED. 2.2
RISTRICTOR VALVE MED. 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

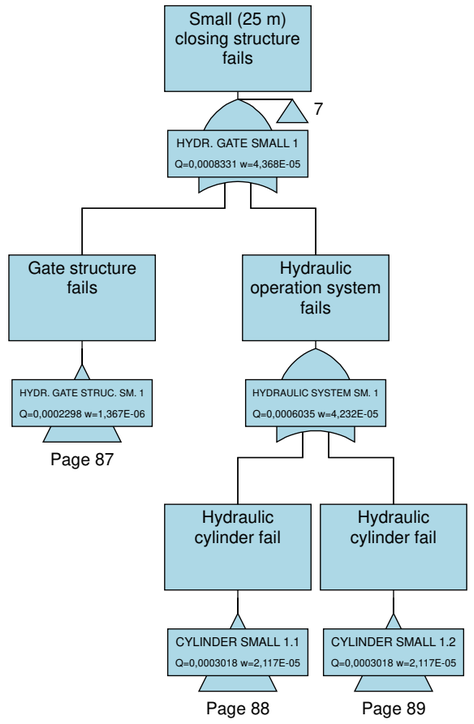

PLC system closing structure
43
PLC 4
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU 4
I/O MODULE 4
POWER MODULE 4
MEMORY MOD 4
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

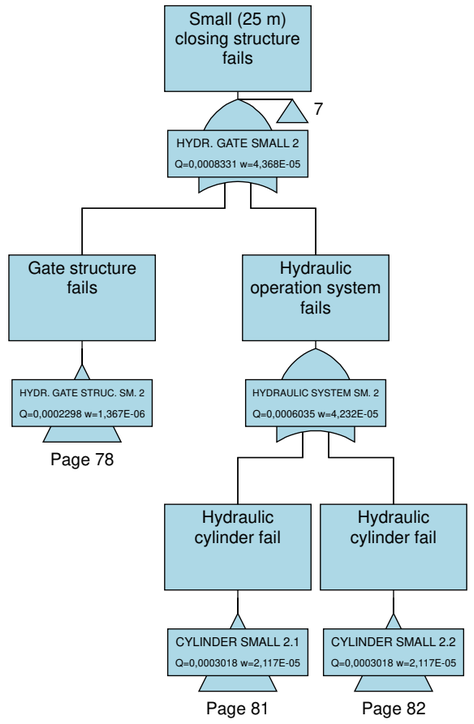

Gate structure fails
44
HYDR. GATE STRUC. LARGE
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. LARGE
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. LARGE
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
HYDR. GATE BEAR. LAR. 1.1
HYDR. GATE BEAR. LAR. 1.2
HYDR. BEAR. BRAC. LAR. 1.1
HYDR. BEAR. BRAC. LAR. 1.2
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

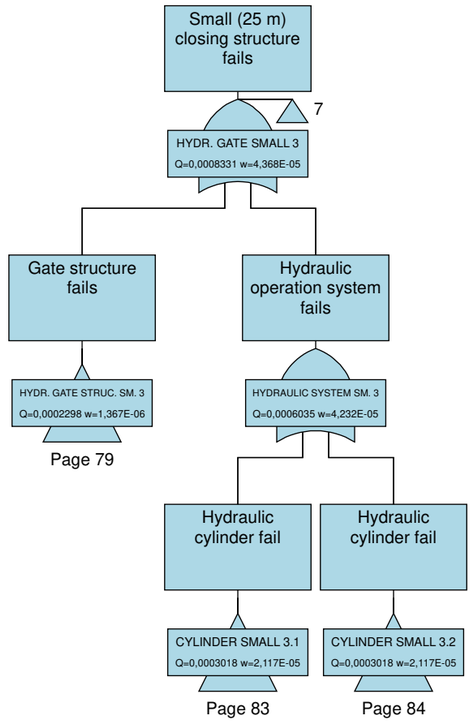

Hydraulic cylinder fail
44
CYLINDER LARGE 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER LAR. 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER LARGE 1.1
BEAR ROD EYE LARGE 1.1
B.T.C. LARGE 1.1
HYDR. LIMIT SW. LARGE 1.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. LAR. 1.1
VALVE BLOCK LARGE 1.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. LARGE 1.1
HYDR. LIMIT SW. LARGE 1.2
DIRECTION VALVE LAR. 1.1
DIFFERENTIAL VALVE LAR. 1.1
PRES. REL. VALVE LAR. 1.1
NON-RETURN VALVE LAR. 1.1
RISTRICTOR VALVE LAR. 1.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

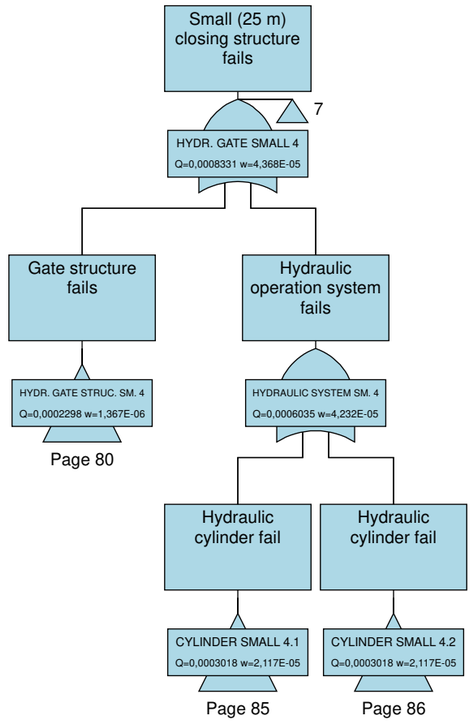

Hydraulic cylinder fail
44
CYLINDER LARGE 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER LAR. 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER LARGE 1.2
BEAR ROD EYE LARGE 1.2
B.T.C. LARGE 1.2
HYDR. LIMIT SW. LARGE 1.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. LAR. 1.2
VALVE BLOCK LARGE 1.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. LARGE 1.3
HYDR. LIMIT SW. LARGE 1.4
DIRECTION VALVE LAR. 1.2
DIFFERENTIAL VALVE LAR. 1.2
PRES. REL. VALVE LAR. 1.2
NON-RETURN VALVE LAR. 1.2
RISTRICTOR VALVE LAR. 1.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

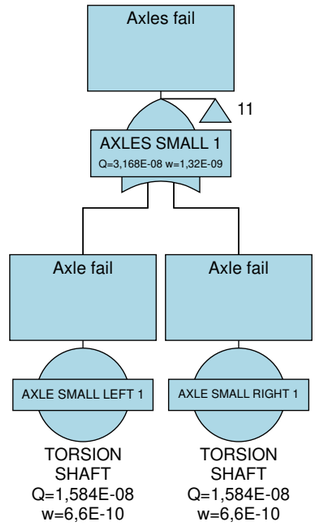

Control system
7
HYDR. CONT. SMALL 1
Q=1,147E-05 w=7,168E-07
Control system
Local control panels
CONTROLS 2
Q=0,00479 w=9,977E-05
LOCAL CONTROL 2
LOCAL CONTROL PAN
Q=0,002394
w=9,976E-05
PLC system closing structure
Software
Control and signal cables
PLC 2
Q=0,004737 w=9,853E-05
SOFTWARE 2
CABLES CONTROL 2
Page 77
SOFTWARE
Q=4,8E-05
w=1E-06
CONTROL CAB
Q=6E-06
w=2,5E-07

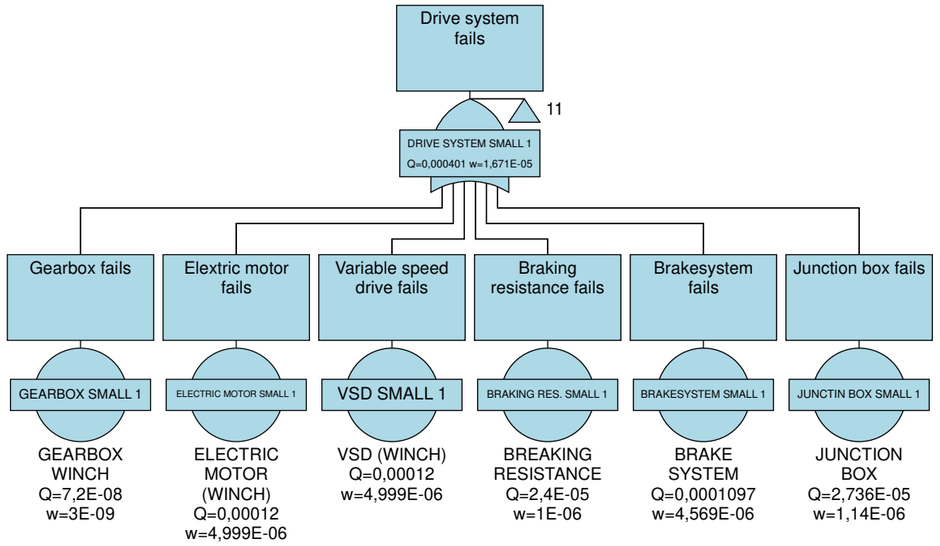

Small (25 m) closing structure fails
7
HYDR. GATE SMALL 1
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. SM. 1
Q=0,0002298 w=1,367E-06
Page 87
HYDRAULIC SYSTEM SM. 1
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER SMALL 1.1
Q=0,0003018 w=2,117E-05
CYLINDER SMALL 1.2
Q=0,0003018 w=2,117E-05
Page 88
Page 89

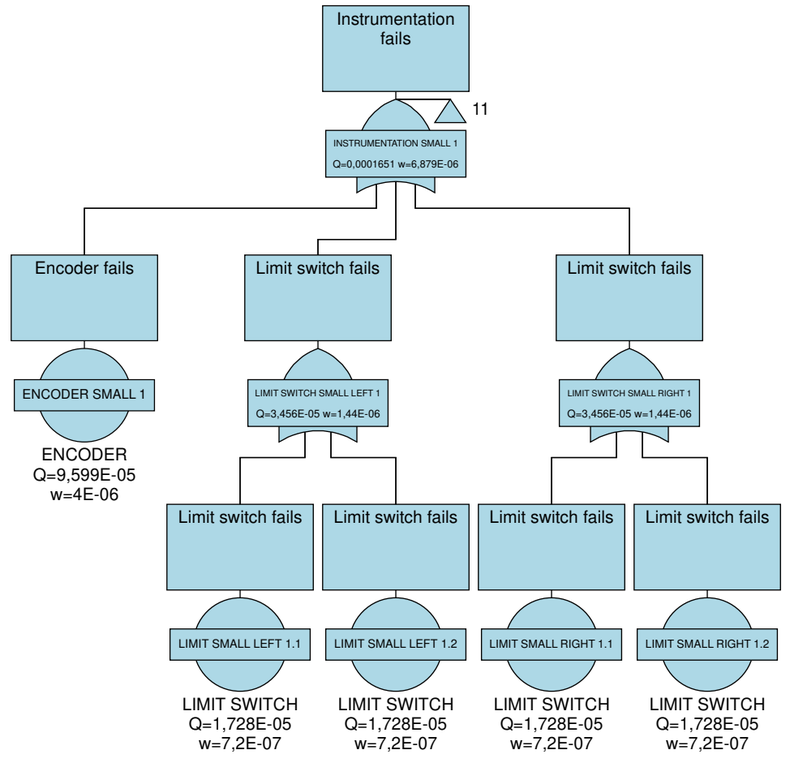

Small (25 m) closing structure fails
7
HYDR. GATE SMALL 2
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. SM. 2
Q=0,0002298 w=1,367E-06
Page 78
HYDRAULIC SYSTEM SM. 2
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER SMALL 2.1
Q=0,0003018 w=2,117E-05
CYLINDER SMALL 2.2
Q=0,0003018 w=2,117E-05
Page 81
Page 82

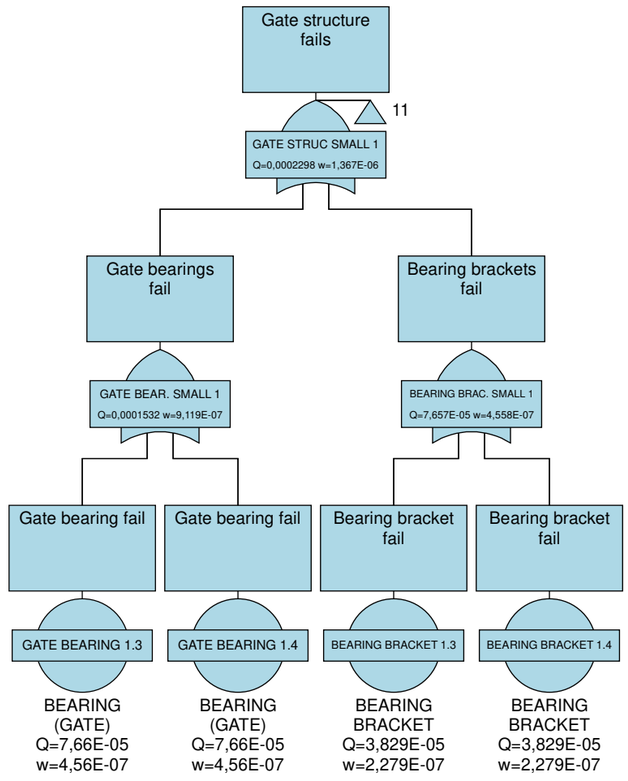

Small (25 m) closing structure fails
7
HYDR. GATE SMALL 3
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. SM. 3
Q=0,0002298 w=1,367E-06
Page 79
HYDRAULIC SYSTEM SM. 3
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER SMALL 3.1
Q=0,0003018 w=2,117E-05
CYLINDER SMALL 3.2
Q=0,0003018 w=2,117E-05
Page 83
Page 84

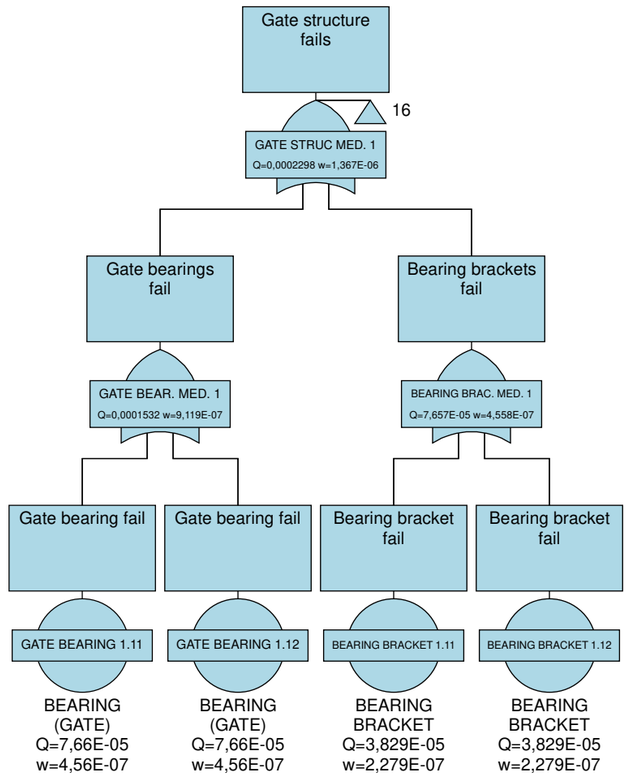

Small (25 m) closing structure fails
7
HYDR. GATE SMALL 4
Q=0,0008331 w=4,368E-05
Gate structure fails
Hydraulic operation system fails
HYDR. GATE STRUC. SM. 4
Q=0,0002298 w=1,367E-06
Page 80
HYDRAULIC SYSTEM SM. 4
Q=0,0006035 w=4,232E-05
Hydraulic cylinder fail
Hydraulic cylinder fail
CYLINDER SMALL 4.1
Q=0,0003018 w=2,117E-05
CYLINDER SMALL 4.2
Q=0,0003018 w=2,117E-05
Page 85
Page 86

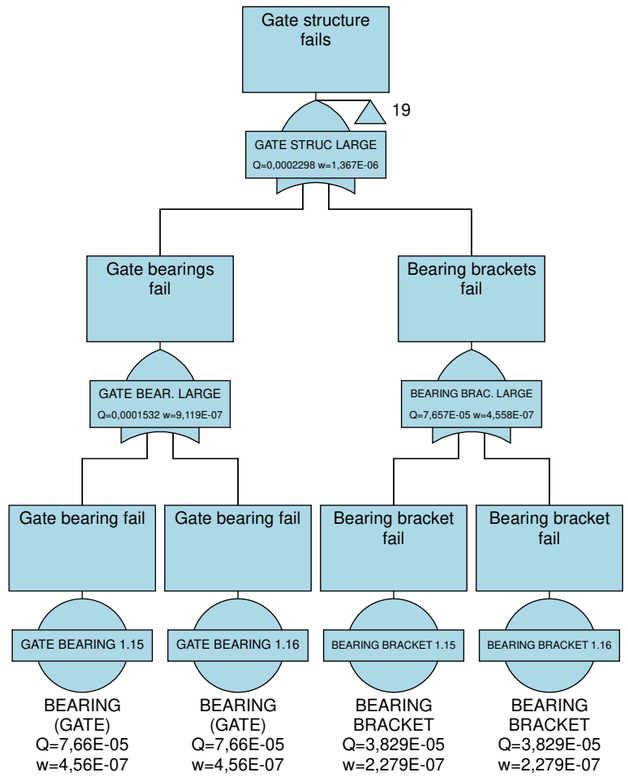

Axles fail
11
AXLES SMALL 1
Q=3,168E-08 w=1,32E-09
Axle fail
Axle fail
AXLE SMALL LEFT 1
AXLE SMALL RIGHT 1
TORSION SHAFT
Q=1,584E-08
w=6,6E-10
TORSION SHAFT
Q=1,584E-08
w=6,6E-10

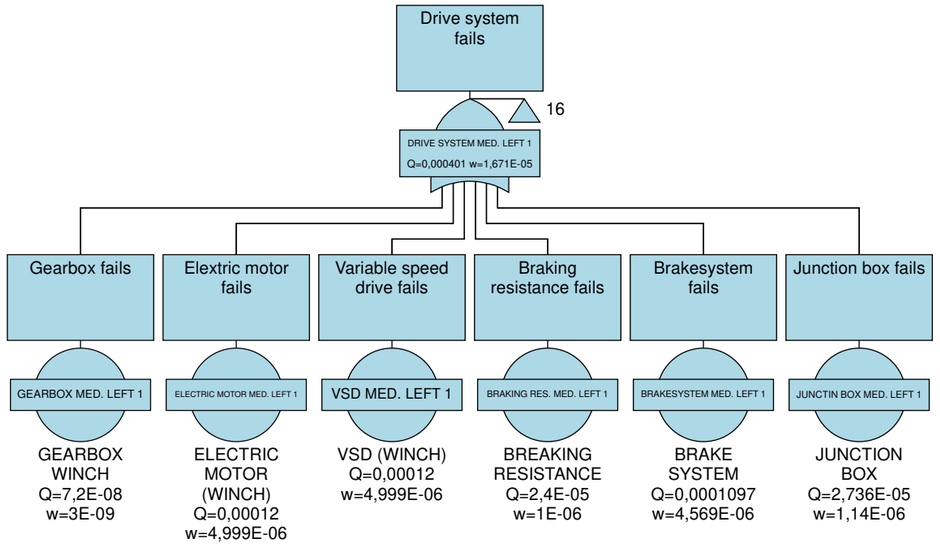

Drive system fails
11
DRIVE SYSTEM SMALL 1
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX SMALL 1
ELECTRIC MOTOR SMALL 1
VSD SMALL 1
BRAKING RES. SMALL 1
BRAKESYSTEM SMALL 1
JUNCTIN BOX SMALL 1
GEARBOX WINCH
Q=7,2E-08
w=3E-09
ELECTRIC MOTOR (WINCH)
Q=0,00012
w=4,999E-06
VSD (WINCH)
Q=0,00012
w=4,999E-06
BREAKING RESISTANCE
Q=2,4E-05
w=1E-06
BRAKE SYSTEM
Q=0,0001097
w=4,569E-06
JUNCTION BOX
Q=2,736E-05
w=1,14E-06

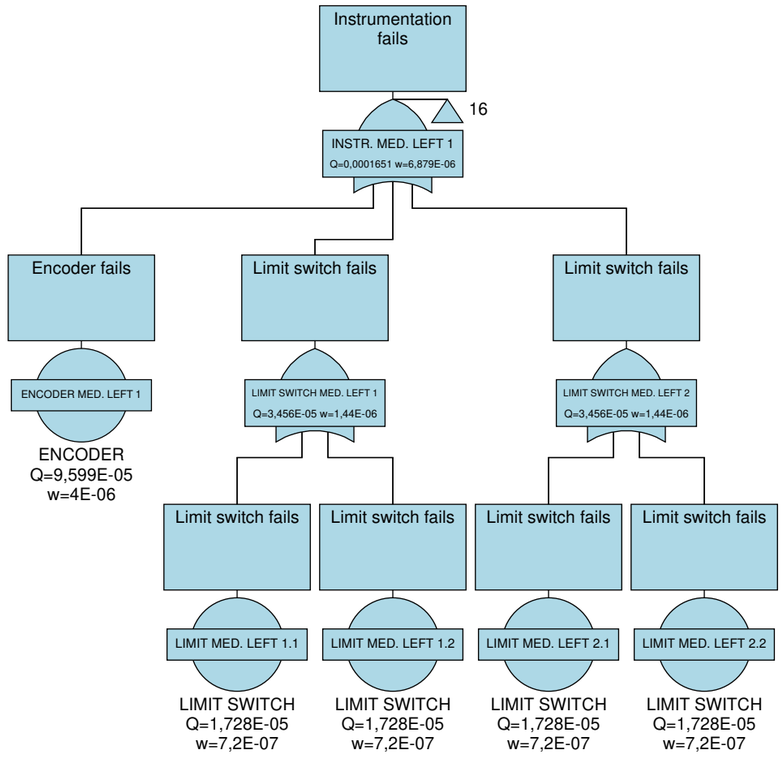

Instrumentation fails
11
INSTRUMENTATION SMALL 1
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER SMALL 1
LIMIT SWITCH SMALL LEFT 1
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH SMALL RIGHT 1
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT SMALL LEFT 1.1
LIMIT SMALL LEFT 1.2
LIMIT SMALL RIGHT 1.1
LIMIT SMALL RIGHT 1.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

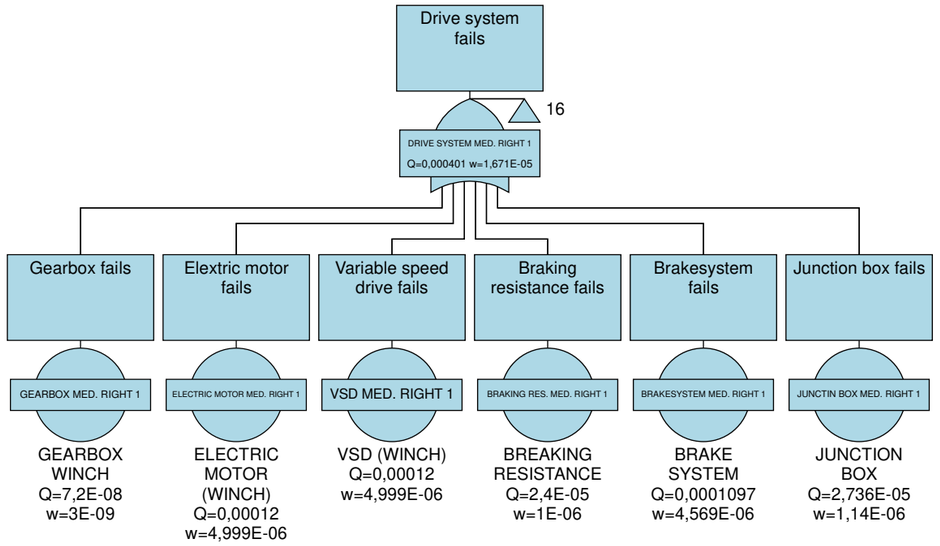

Gate structure fails
11
GATE STRUC SMALL 1
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. SMALL 1
Q=0,0001532 w=9,119E-07
BEARING BRAC. SMALL 1
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.3
GATE BEARING 1.4
BEARING BRACKET 1.3
BEARING BRACKET 1.4
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

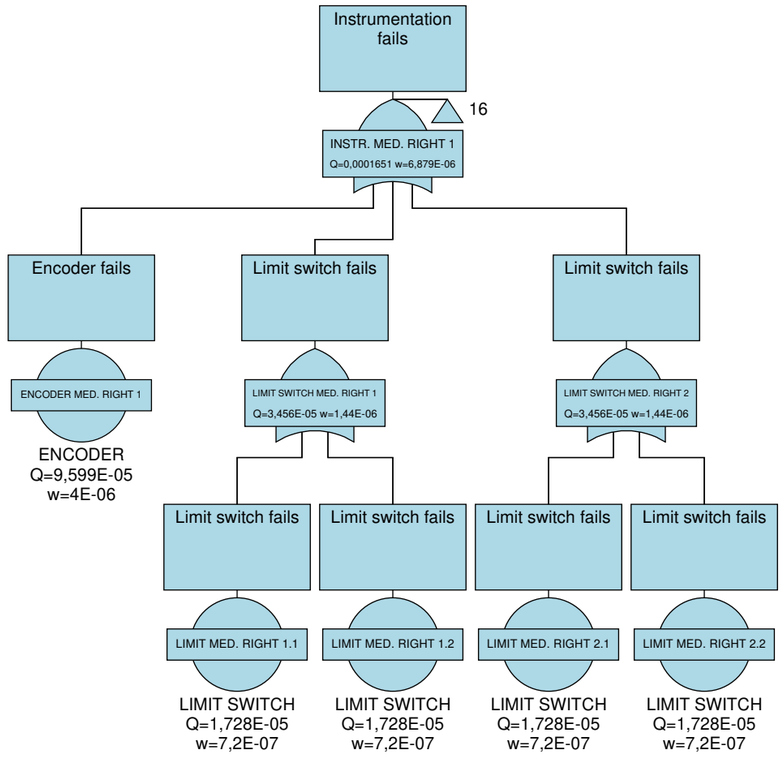

Gate structure fails
16
GATE STRUC MED. 1
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. MED. 1
Q=0,0001532 w=9,119E-07
BEARING BRAC. MED. 1
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.11
GATE BEARING 1.12
BEARING BRACKET 1.11
BEARING BRACKET 1.12
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

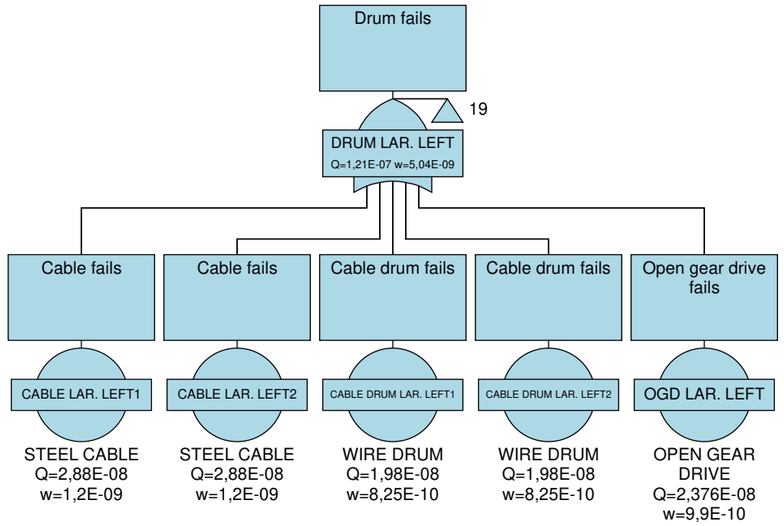

Gate structure fails
19
GATE STRUC LARGE
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
GATE BEAR. LARGE
Q=0,0001532 w=9,119E-07
BEARING BRAC. LARGE
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.15
GATE BEARING 1.16
BEARING BRACKET 1.15
BEARING BRACKET 1.16
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

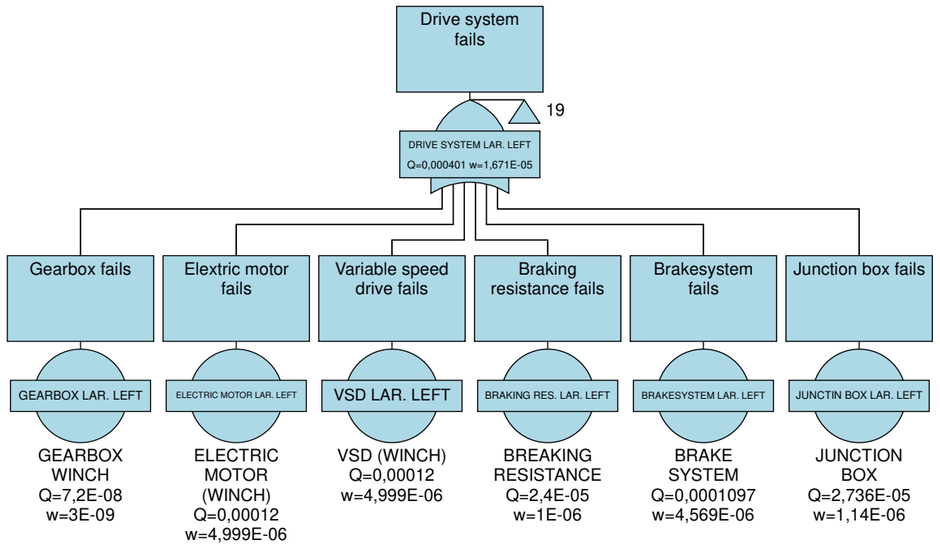

Drive system fails
16
DRIVE SYSTEM MED. LEFT 1
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX MED. LEFT 1
ELECTRIC MOTOR MED. LEFT 1
VSD MED. LEFT 1
BRAKING RES. MED. LEFT 1
BRAKESYSTEM MED. LEFT 1
JUNCTIN BOX MED. LEFT 1
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

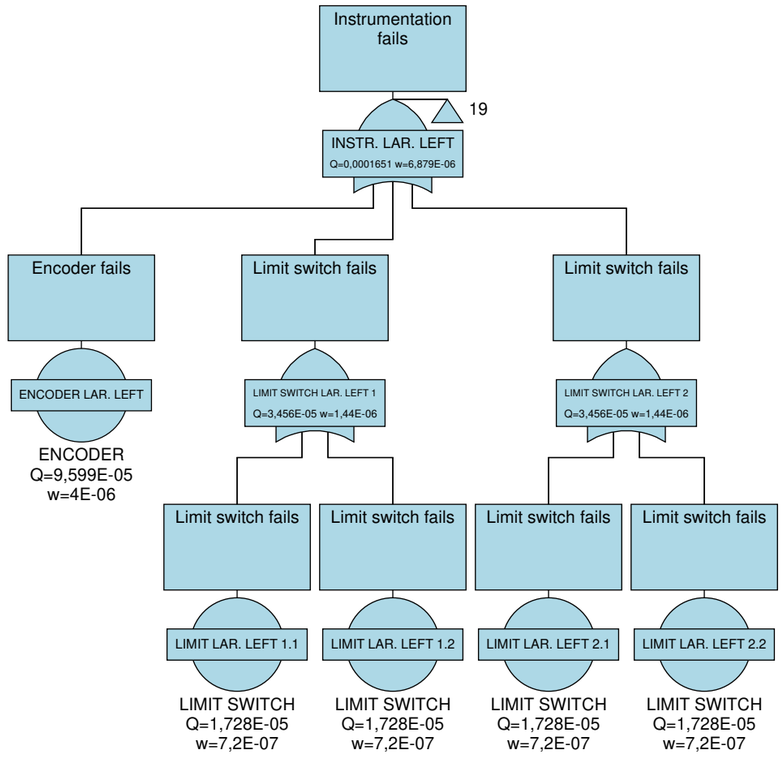

Instrumentation fails
16
INSTR. MED. LEFT 1
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER MED. LEFT 1
LIMIT SWITCH MED. LEFT 1
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH MED. LEFT 2
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT MED. LEFT 1.1
LIMIT MED. LEFT 1.2
LIMIT MED. LEFT 2.1
LIMIT MED. LEFT 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

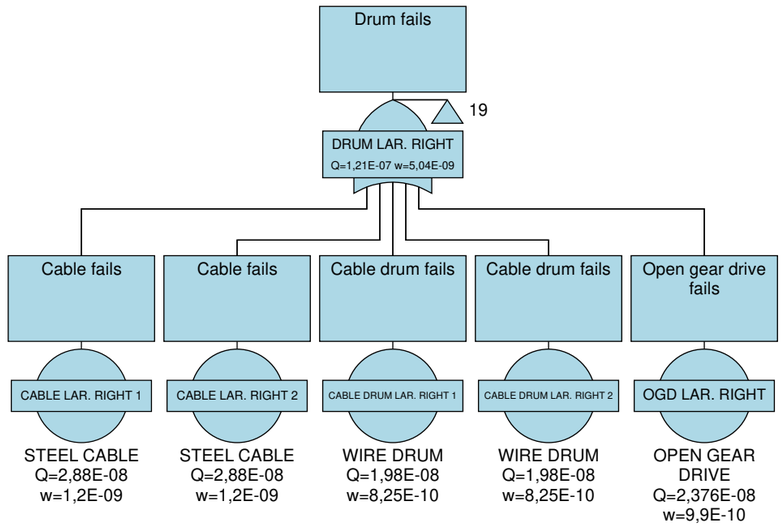

Drive system fails
16
DRIVE SYSTEM MED. RIGHT 1
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX MED. RIGHT 1
ELECTRIC MOTOR MED. RIGHT 1
VSD MED. RIGHT 1
BRAKING RES. MED. RIGHT 1
BRAKESYSTEM MED. RIGHT 1
JUNCTIN BOX MED. RIGHT 1
GEARBOX WINCH
Q=7,2E-08
w=3E-09
ELECTRIC MOTOR (WINCH)
Q=0,00012
w=4,999E-06
VSD (WINCH)
Q=0,00012
w=4,999E-06
BREAKING RESISTANCE
Q=2,4E-05
w=1E-06
BRAKE SYSTEM
Q=0,0001097
w=4,569E-06
JUNCTION BOX
Q=2,736E-05
w=1,14E-06

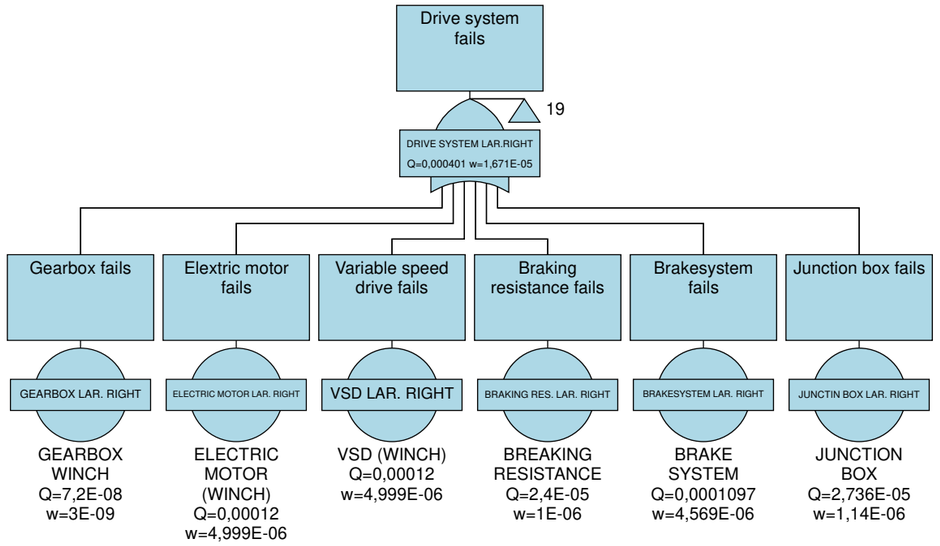

Instrumentation fails
16
INSTR. MED. RIGHT 1
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER MED. RIGHT 1
LIMIT SWITCH MED. RIGHT 1
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH MED. RIGHT 2
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT MED. RIGHT 1.1
LIMIT MED. RIGHT 1.2
LIMIT MED. RIGHT 2.1
LIMIT MED. RIGHT 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

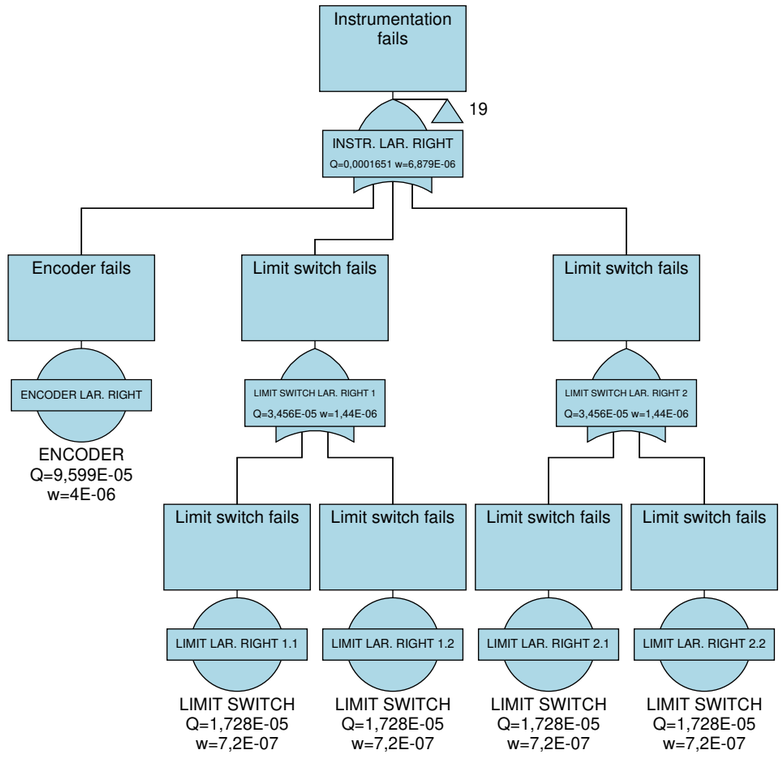

Drum fails
19
DRUM LAR. LEFT
Q=1,21E-07 w=5,04E-09
Cable fails
Cable fails
Cable drum fails
Cable drum fails
Open gear drive fails
CABLE LAR. LEFT1
CABLE LAR. LEFT2
CABLE DRUM LAR. LEFT1
CABLE DRUM LAR. LEFT2
OGD LAR. LEFT
STEEL CABLE
Q=2,88E-08
w=1,2E-09
STEEL CABLE
Q=2,88E-08
w=1,2E-09
WIRE DRUM
Q=1,98E-08
w=8,25E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10

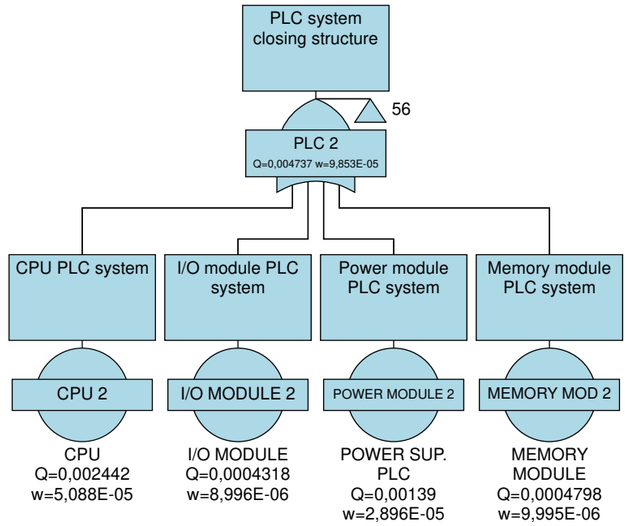

Drive system fails
19
DRIVE SYSTEM LAR. LEFT
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX LAR. LEFT
ELECTRIC MOTOR LAR. LEFT
VSD LAR. LEFT
BRAKING RES. LAR. LEFT
BRAKESYSTEM LAR. LEFT
JUNCTIN BOX LAR. LEFT
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

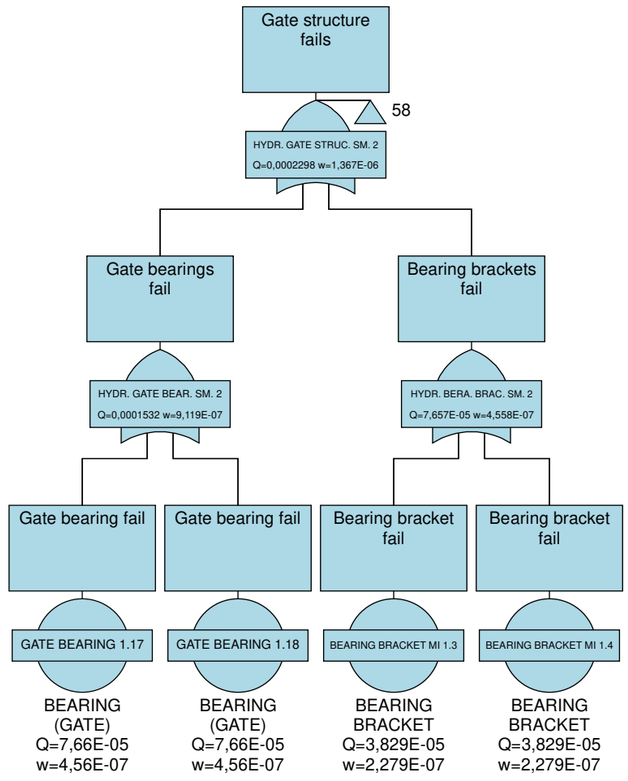

Instrumentation fails
19
INSTR. LAR. LEFT
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER LAR. LEFT
LIMIT SWITCH LAR. LEFT 1
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH LAR. LEFT 2
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT LAR. LEFT 1.1
LIMIT LAR. LEFT 1.2
LIMIT LAR. LEFT 2.1
LIMIT LAR. LEFT 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

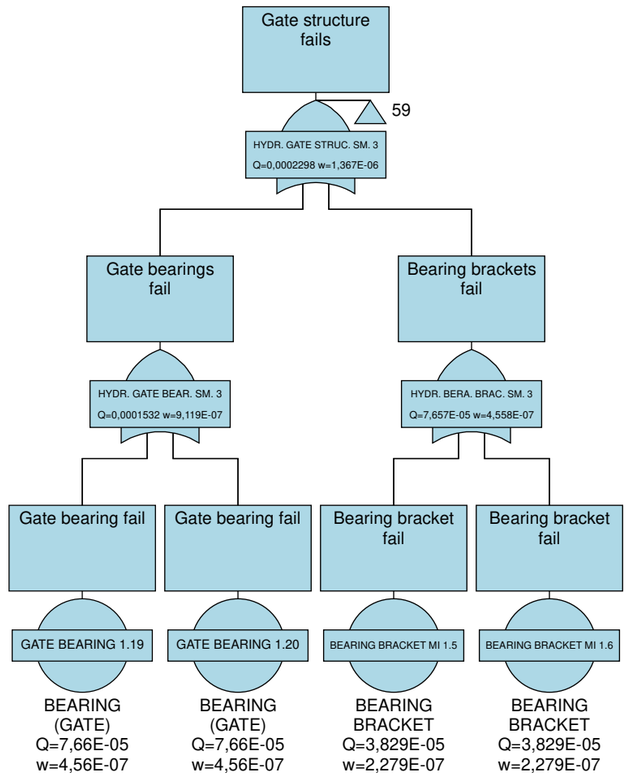

Drum fails
19
DRUM LAR. RIGHT
Q=1,21E-07 w=5,04E-09
Cable fails
Cable fails
Cable drum fails
Cable drum fails
Open gear drive fails
CABLE LAR. RIGHT 1
CABLE LAR. RIGHT 2
CABLE DRUM LAR. RIGHT 1
CABLE DRUM LAR. RIGHT 2
OGD LAR. RIGHT
STEEL CABLE
Q=2,88E-08
w=1,2E-09
STEEL CABLE
Q=2,88E-08
w=1,2E-09
WIRE DRUM
Q=1,98E-08
w=8,25E-10
WIRE DRUM
Q=1,98E-08
w=8,25E-10
OPEN GEAR DRIVE
Q=2,376E-08
w=9,9E-10

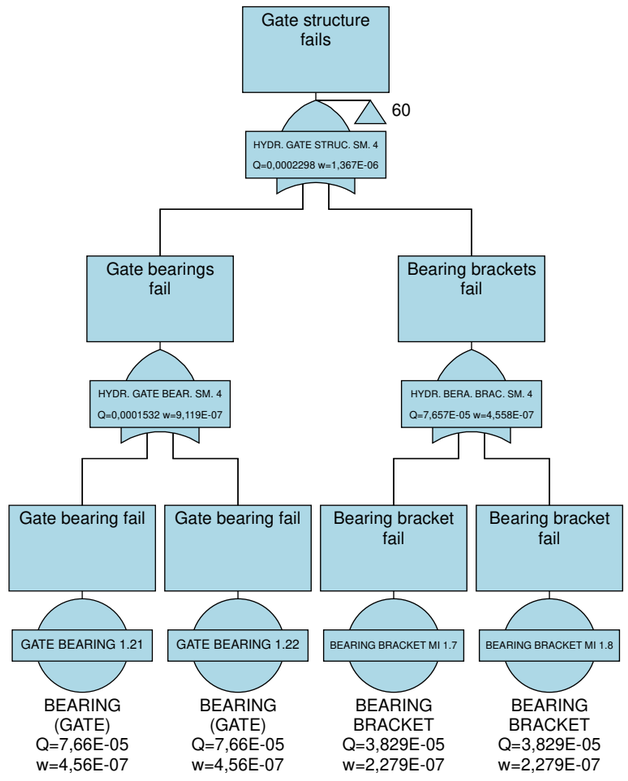

Drive system fails
19
DRIVE SYSTEM LAR.RIGHT
Q=0,000401 w=1,671E-05
Gearbox fails
Elextric motor fails
Variable speed drive fails
Braking resistance fails
Brakesystem fails
Junction box fails
GEARBOX LAR. RIGHT
ELECTRIC MOTOR LAR. RIGHT
VSD LAR. RIGHT
BRAKING RES. LAR. RIGHT
BRAKESYSTEM LAR. RIGHT
JUNCTIN BOX LAR. RIGHT
GEARBOX WINCH Q=7,2E-08 w=3E-09
ELECTRIC MOTOR (WINCH) Q=0,00012 w=4,999E-06
VSD (WINCH) Q=0,00012 w=4,999E-06
BREAKING RESISTANCE Q=2,4E-05 w=1E-06
BRAKE SYSTEM Q=0,0001097 w=4,569E-06
JUNCTION BOX Q=2,736E-05 w=1,14E-06

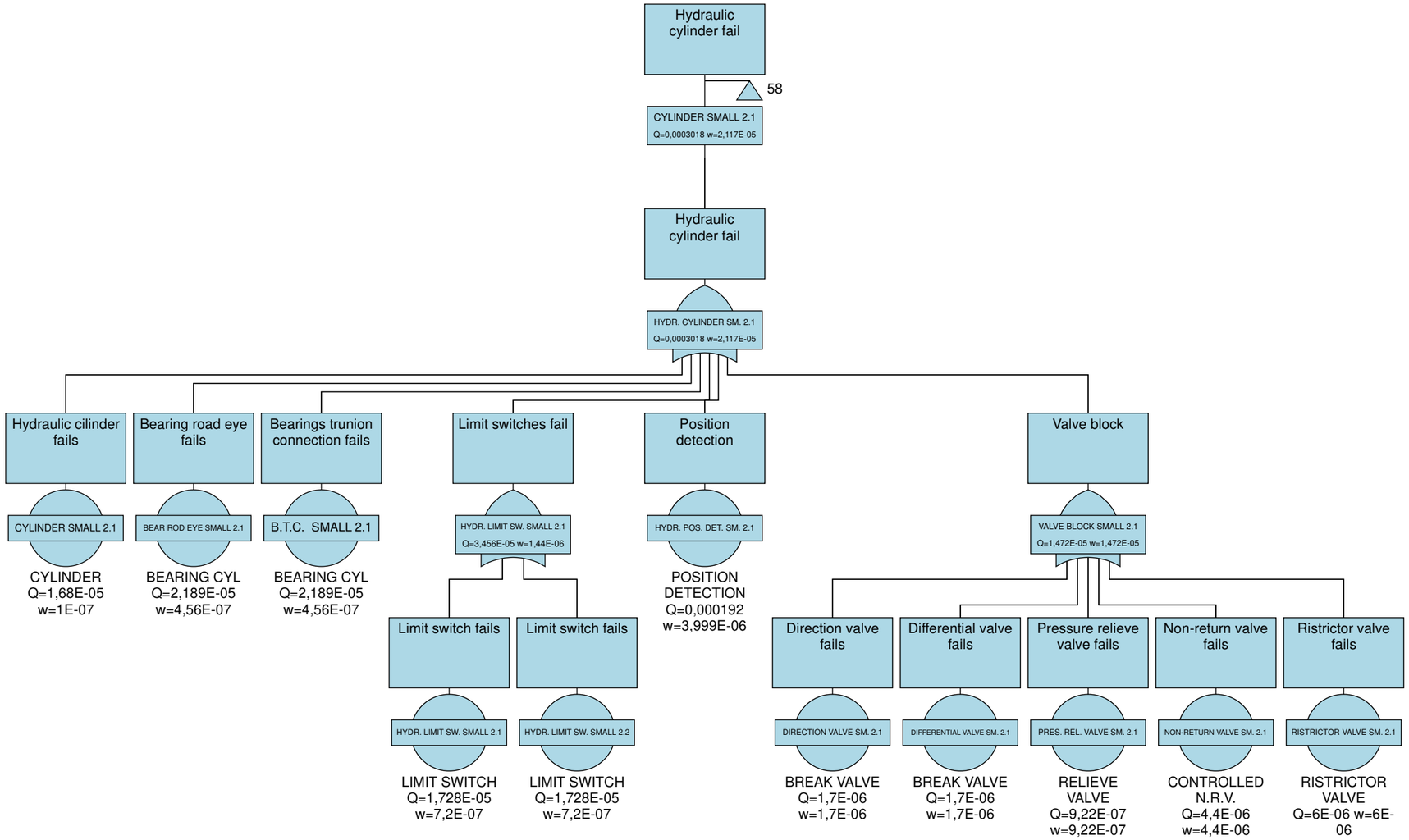

Instrumentation fails
19
INSTR. LAR. RIGHT
Q=0,0001651 w=6,879E-06
Encoder fails
Limit switch fails
Limit switch fails
ENCODER LAR. RIGHT
LIMIT SWITCH LAR. RIGHT 1
Q=3,456E-05 w=1,44E-06
LIMIT SWITCH LAR. RIGHT 2
Q=3,456E-05 w=1,44E-06
ENCODER
Q=9,599E-05
w=4E-06
Limit switch fails
Limit switch fails
Limit switch fails
Limit switch fails
LIMIT LAR. RIGHT 1.1
LIMIT LAR. RIGHT 1.2
LIMIT LAR. RIGHT 2.1
LIMIT LAR. RIGHT 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07

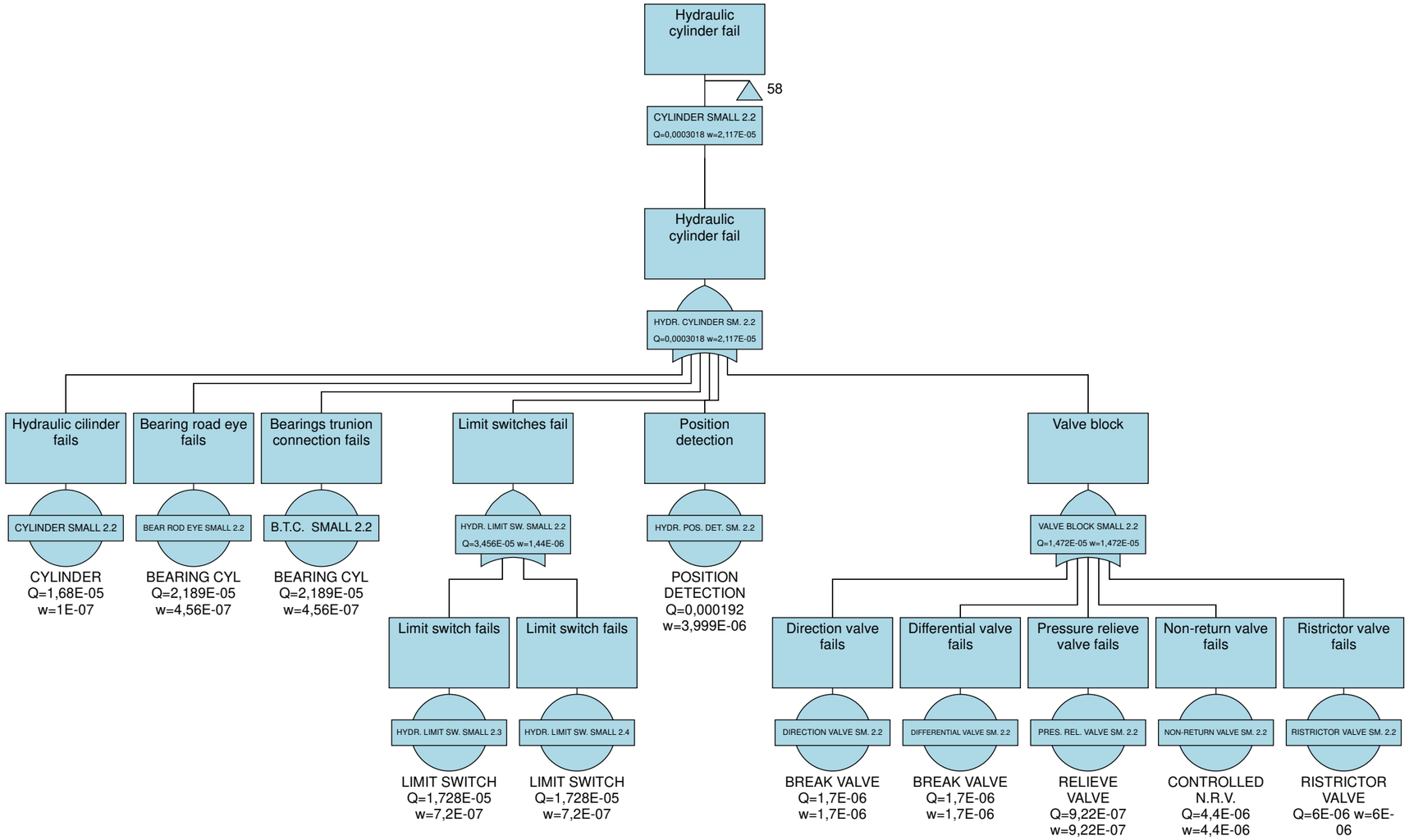

PLC system closing structure
56
PLC 2
Q=0,004737 w=9,853E-05
CPU PLC system
I/O module PLC system
Power module PLC system
Memory module PLC system
CPU 2
I/O MODULE 2
POWER MODULE 2
MEMORY MOD 2
CPU
Q=0,002442
w=5,088E-05
I/O MODULE
Q=0,0004318
w=8,996E-06
POWER SUP. PLC
Q=0,00139
w=2,896E-05
MEMORY MODULE
Q=0,0004798
w=9,995E-06

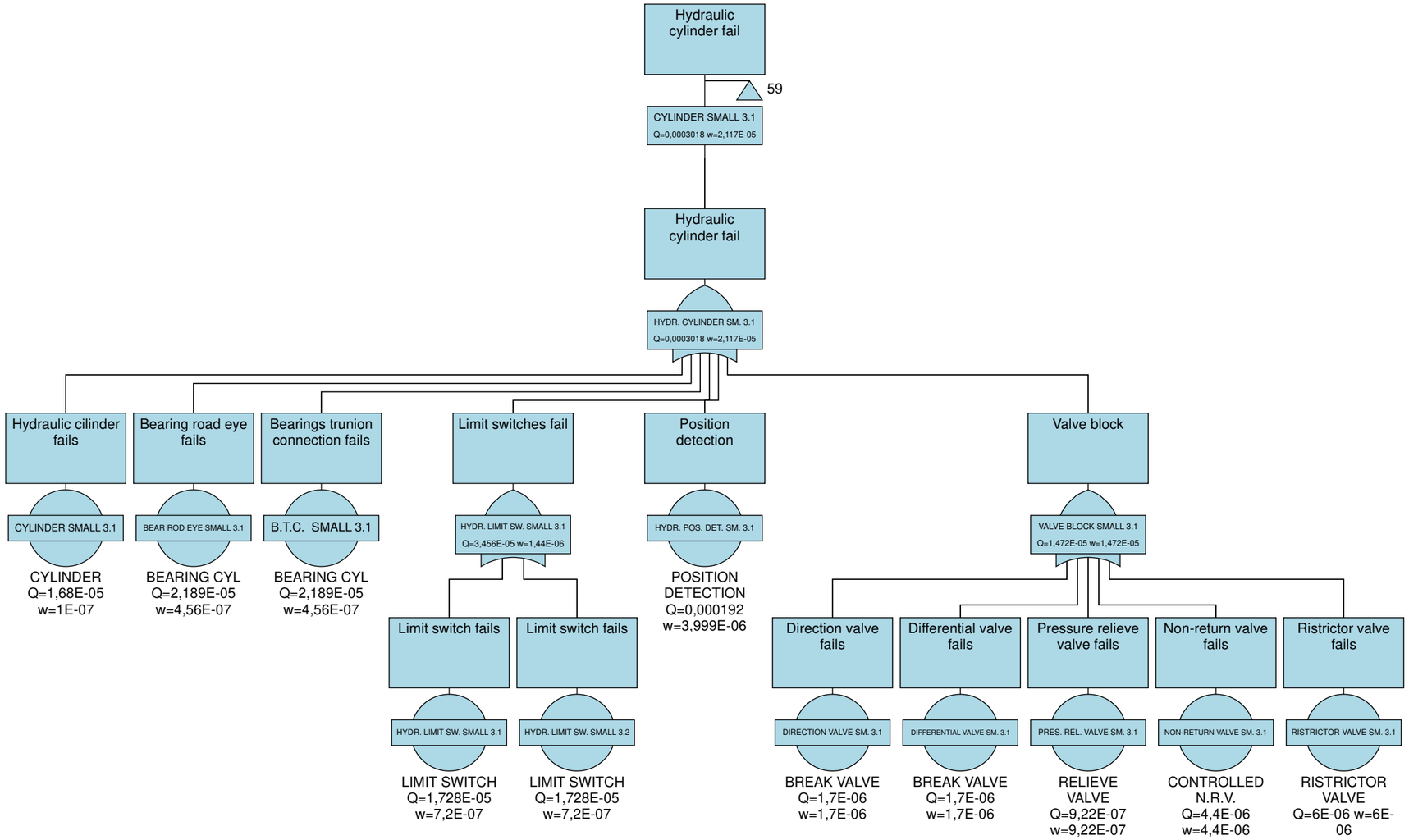

Gate structure fails
58
HYDR. GATE STRUC. SM. 2
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. SM. 2
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. SM. 2
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.17
GATE BEARING 1.18
BEARING BRACKET MI 1.3
BEARING BRACKET MI 1.4
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

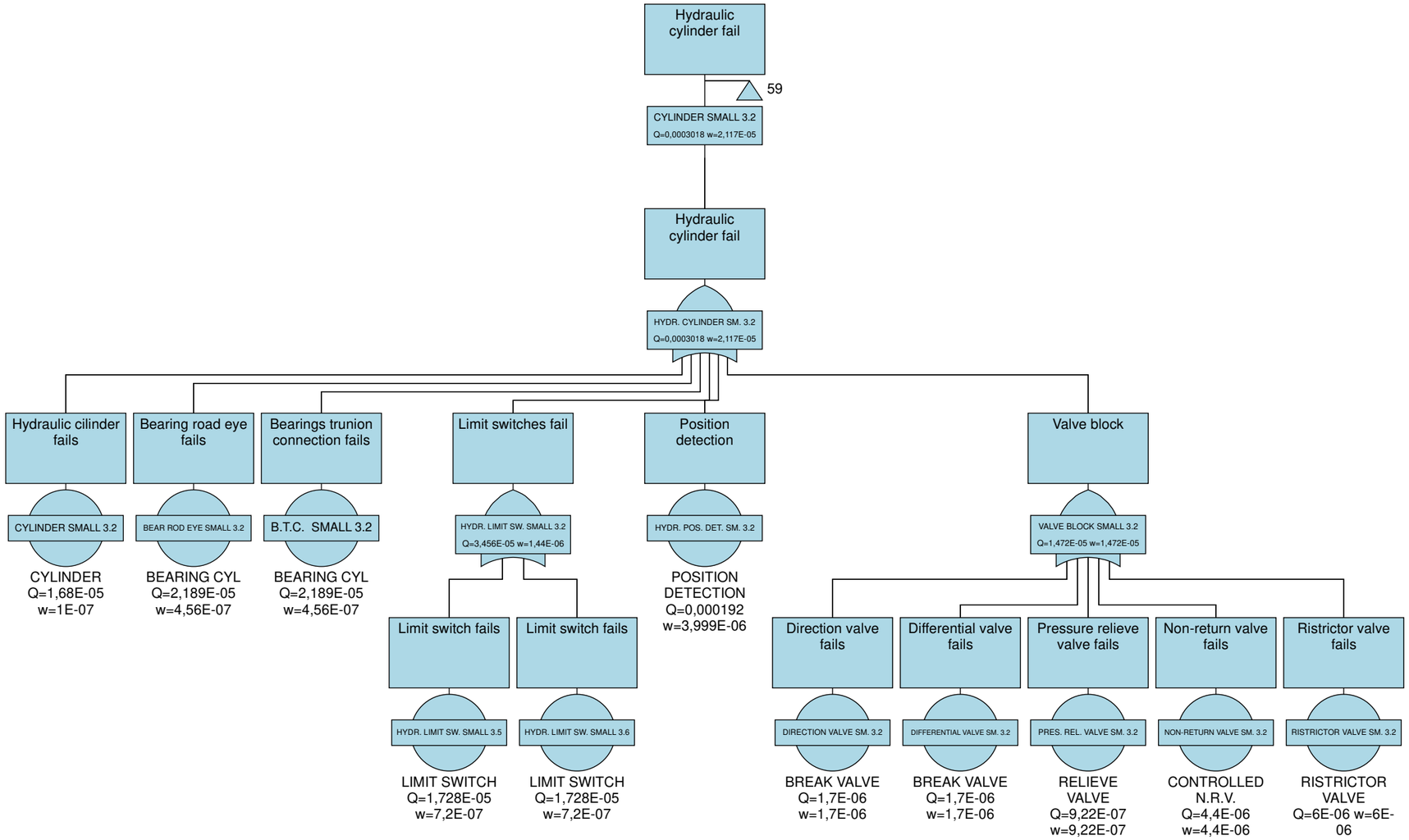

Gate structure fails
59
HYDR. GATE STRUC. SM. 3
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. SM. 3
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. SM. 3
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.19
GATE BEARING 1.20
BEARING BRACKET MI 1.5
BEARING BRACKET MI 1.6
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

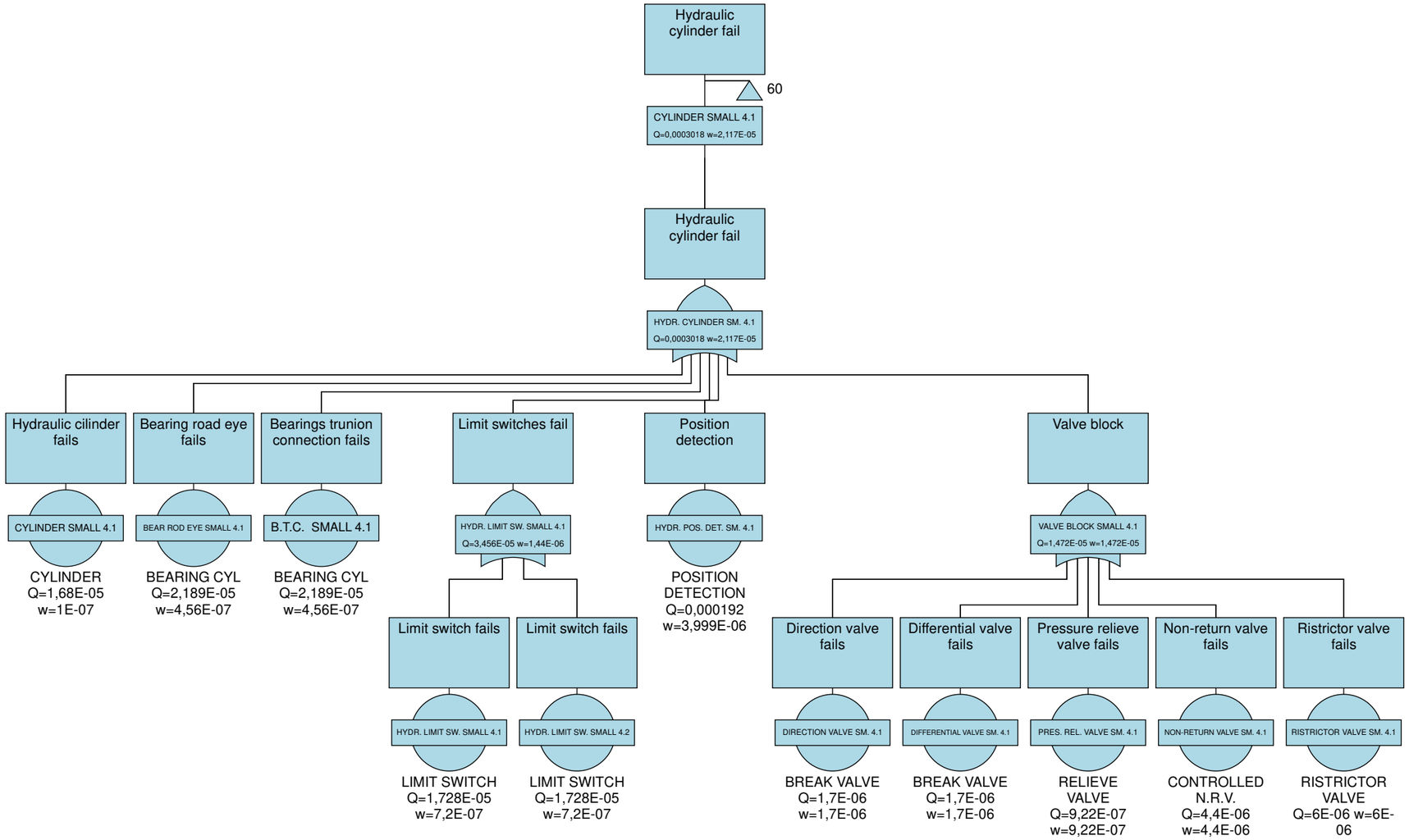

Gate structure fails
60
HYDR. GATE STRUC. SM. 4
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. SM. 4
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. SM. 4
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.21
GATE BEARING 1.22
BEARING BRACKET MI 1.7
BEARING BRACKET MI 1.8
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07

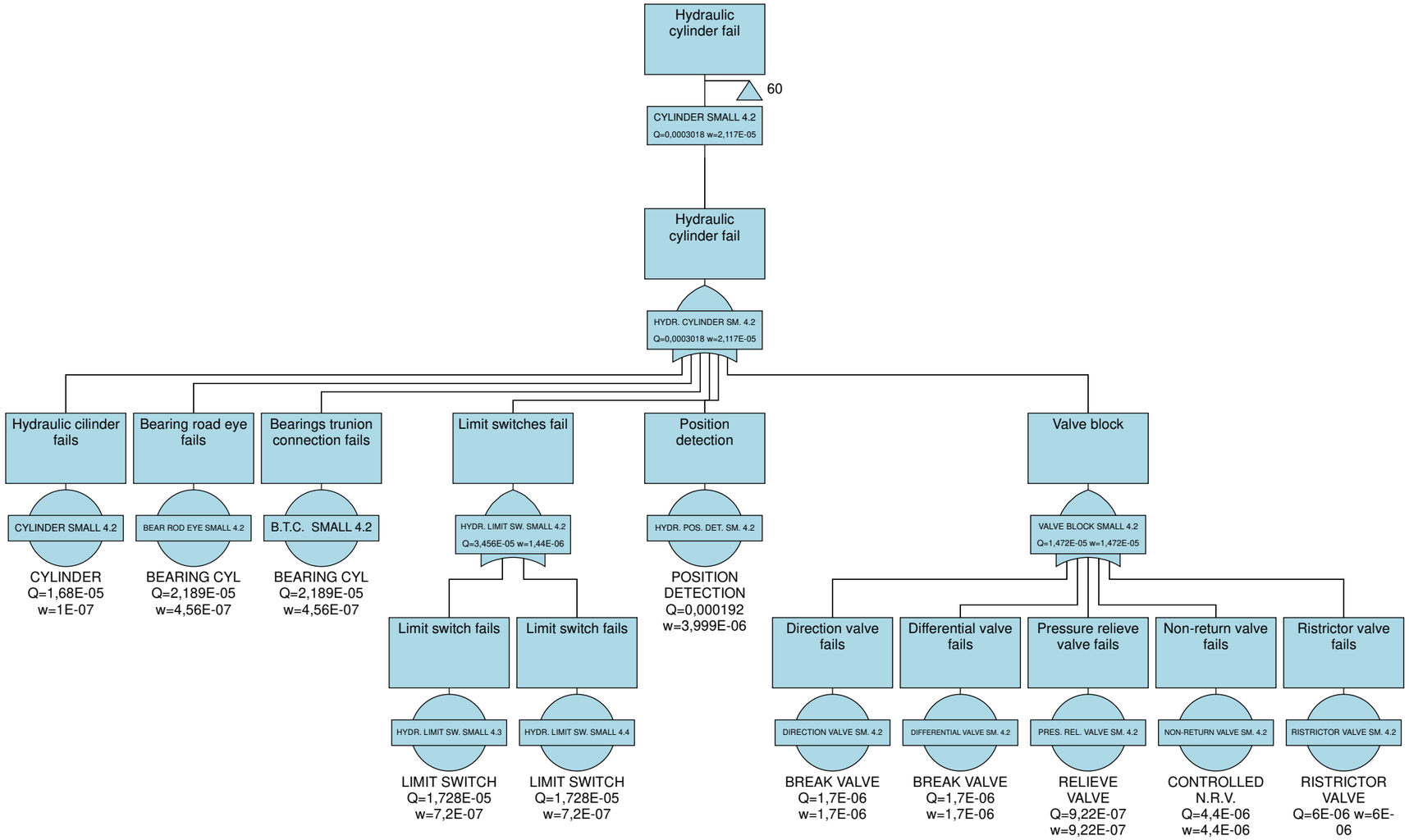

Hydraulic cylinder fail
58
CYLINDER SMALL 2.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 2.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 2.1
BEAR ROD EYE SMALL 2.1
B.T.C. SMALL 2.1
HYDR. LIMIT SW. SMALL 2.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 2.1
VALVE BLOCK SMALL 2.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 2.1
HYDR. LIMIT SW. SMALL 2.2
DIRECTION VALVE SM. 2.1
DIFFERENTIAL VALVE SM. 2.1
PRES. REL. VALVE SM. 2.1
NON-RETURN VALVE SM. 2.1
RISTRICTOR VALVE SM. 2.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

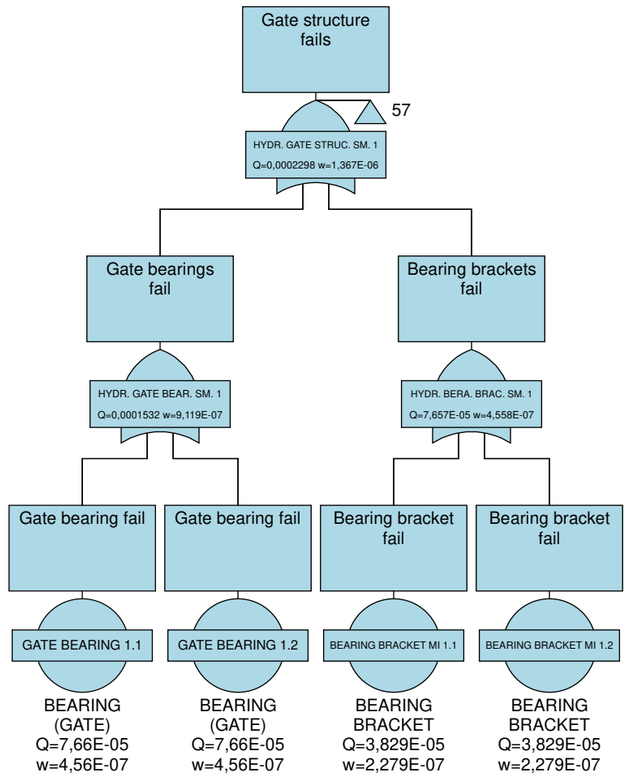

Hydraulic cylinder fail
58
CYLINDER SMALL 2.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 2.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 2.2
BEAR ROD EYE SMALL 2.2
B.T.C. SMALL 2.2
HYDR. LIMIT SW. SMALL 2.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 2.2
VALVE BLOCK SMALL 2.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 2.3
HYDR. LIMIT SW. SMALL 2.4
DIRECTION VALVE SM. 2.2
DIFFERENTIAL VALVE SM. 2.2
PRES. REL. VALVE SM. 2.2
NON-RETURN VALVE SM. 2.2
RISTRICTOR VALVE SM. 2.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

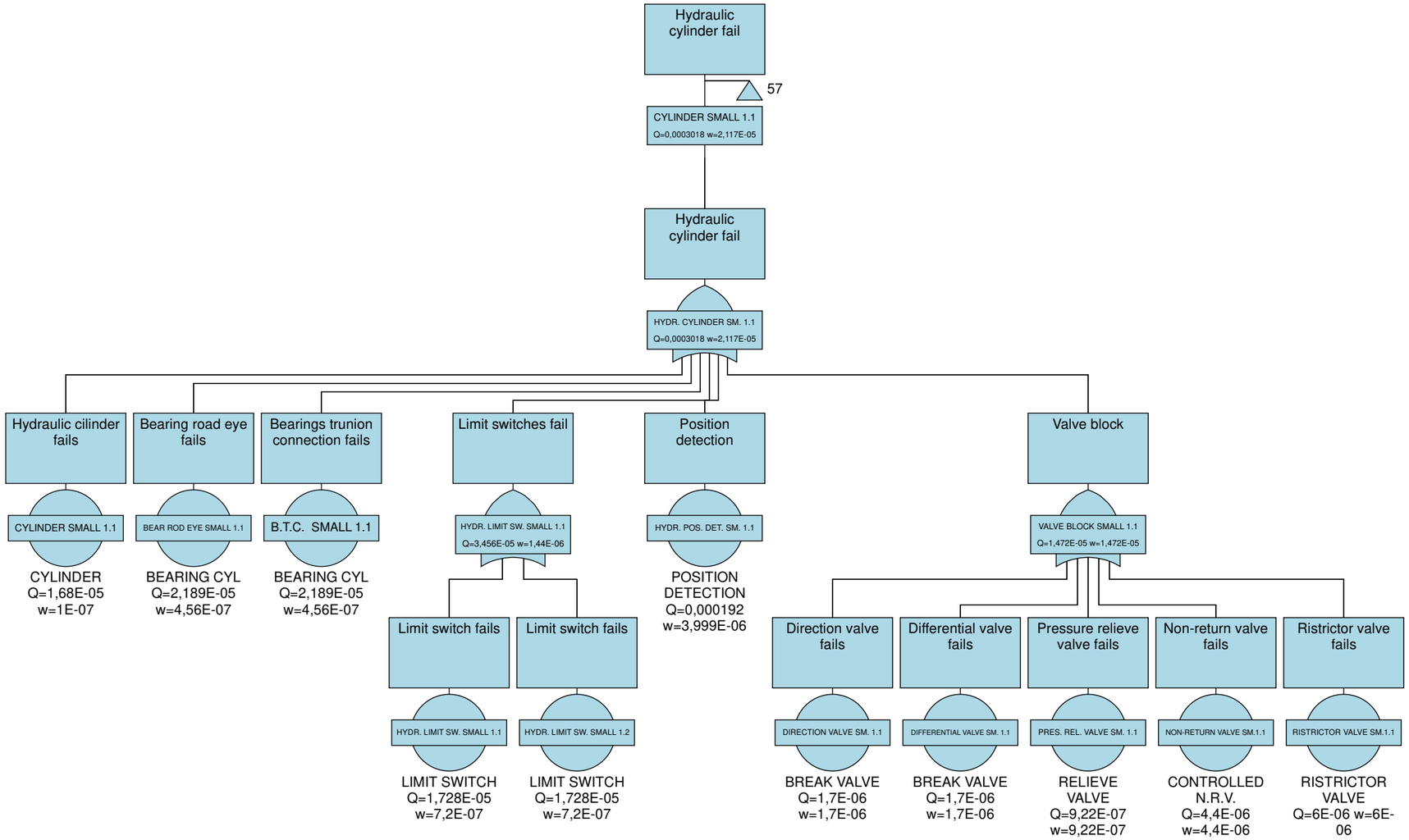

Hydraulic cylinder fail
59
CYLINDER SMALL 3.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 3.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 3.1
BEAR ROD EYE SMALL 3.1
B.T.C. SMALL 3.1
HYDR. LIMIT SW. SMALL 3.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 3.1
VALVE BLOCK SMALL 3.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 3.1
HYDR. LIMIT SW. SMALL 3.2
DIRECTION VALVE SM. 3.1
DIFFERENTIAL VALVE SM. 3.1
PRES. REL. VALVE SM. 3.1
NON-RETURN VALVE SM. 3.1
RISTRICTOR VALVE SM. 3.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

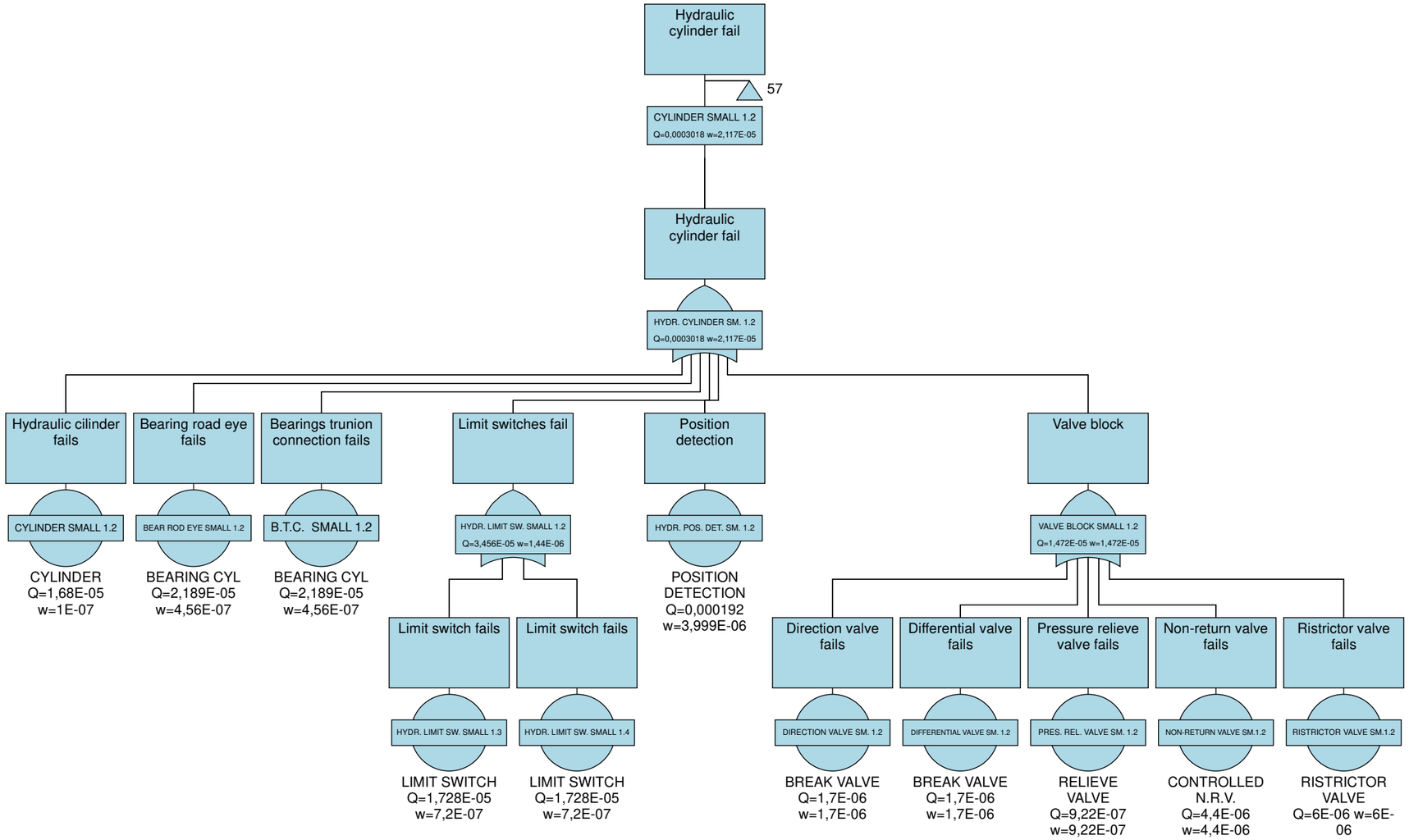

Hydraulic cylinder fail
59
CYLINDER SMALL 3.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 3.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 3.2
BEAR ROD EYE SMALL 3.2
B.T.C. SMALL 3.2
HYDR. LIMIT SW. SMALL 3.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 3.2
VALVE BLOCK SMALL 3.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 3.5
HYDR. LIMIT SW. SMALL 3.6
DIRECTION VALVE SM. 3.2
DIFFERENTIAL VALVE SM. 3.2
PRES. REL. VALVE SM. 3.2
NON-RETURN VALVE SM. 3.2
RISTRICTOR VALVE SM. 3.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06


Hydraulic cylinder fail
60
CYLINDER SMALL 4.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 4.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 4.1
BEAR ROD EYE SMALL 4.1
B.T.C. SMALL 4.1
HYDR. LIMIT SW. SMALL 4.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 4.1
VALVE BLOCK SMALL 4.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 4.1
HYDR. LIMIT SW. SMALL 4.2
DIRECTION VALVE SM. 4.1
DIFFERENTIAL VALVE SM. 4.1
PRES. REL. VALVE SM. 4.1
NON-RETURN VALVE SM. 4.1
RISTRICTOR VALVE SM. 4.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06


Hydraulic cylinder fail
60
CYLINDER SMALL 4.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 4.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 4.2
BEAR ROD EYE SMALL 4.2
B.T.C. SMALL 4.2
HYDR. LIMIT SW. SMALL 4.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 4.2
VALVE BLOCK SMALL 4.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 4.3
HYDR. LIMIT SW. SMALL 4.4
DIRECTION VALVE SM. 4.2
DIFFERENTIAL VALVE SM. 4.2
PRES. REL. VALVE SM. 4.2
NON-RETURN VALVE SM. 4.2
RISTRICTOR VALVE SM. 4.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06


Gate structure fails
57
HYDR. GATE STRUC. SM. 1
Q=0,0002298 w=1,367E-06
Gate bearings fail
Bearing brackets fail
HYDR. GATE BEAR. SM. 1
Q=0,0001532 w=9,119E-07
HYDR. BERA. BRAC. SM. 1
Q=7,657E-05 w=4,558E-07
Gate bearing fail
Gate bearing fail
Bearing bracket fail
Bearing bracket fail
GATE BEARING 1.1
GATE BEARING 1.2
BEARING BRACKET MI 1.1
BEARING BRACKET MI 1.2
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING (GATE) Q=7,66E-05 w=4,56E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07
BEARING BRACKET Q=3,829E-05 w=2,279E-07


Hydraulic cylinder fail
57
CYLINDER SMALL 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 1.1
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 1.1
BEAR ROD EYE SMALL 1.1
B.T.C. SMALL 1.1
HYDR. LIMIT SW. SMALL 1.1
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 1.1
VALVE BLOCK SMALL 1.1
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 1.1
HYDR. LIMIT SW. SMALL 1.2
DIRECTION VALVE SM. 1.1
DIFFERENTIAL VALVE SM. 1.1
PRES. REL. VALVE SM. 1.1
NON-RETURN VALVE SM.1.1
RISTRICTOR VALVE SM.1.1
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06


Hydraulic cylinder fail
57
CYLINDER SMALL 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cylinder fail
HYDR. CYLINDER SM. 1.2
Q=0,0003018 w=2,117E-05
Hydraulic cilinder fails
Bearing road eye fails
Bearings trunion connection fails
Limit switches fail
Position detection
Valve block
CYLINDER SMALL 1.2
BEAR ROD EYE SMALL 1.2
B.T.C. SMALL 1.2
HYDR. LIMIT SW. SMALL 1.2
Q=3,456E-05 w=1,44E-06
HYDR. POS. DET. SM. 1.2
VALVE BLOCK SMALL 1.2
Q=1,472E-05 w=1,472E-05
CYLINDER
Q=1,68E-05
w=1E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
BEARING CYL
Q=2,189E-05
w=4,56E-07
Limit switch fails
Limit switch fails
POSITION DETECTION
Q=0,000192
w=3,999E-06
Direction valve fails
Differential valve fails
Pressure relieve valve fails
Non-return valve fails
Ristrictor valve fails
HYDR. LIMIT SW. SMALL 1.3
HYDR. LIMIT SW. SMALL 1.4
DIRECTION VALVE SM. 1.2
DIFFERENTIAL VALVE SM. 1.2
PRES. REL. VALVE SM. 1.2
NON-RETURN VALVE SM.1.2
RISTRICTOR VALVE SM.1.2
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
LIMIT SWITCH
Q=1,728E-05
w=7,2E-07
BREAK VALVE
Q=1,7E-06
w=1,7E-06
BREAK VALVE
Q=1,7E-06
w=1,7E-06
RELIEVE VALVE
Q=9,22E-07
w=9,22E-07
CONTROLLED N.R.V.
Q=4,4E-06
w=4,4E-06
RISTRICTOR VALVE
Q=6E-06 w=6E-06

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
ATS BACK-UP	Rate-MTTR	8E-06	24	1	Automatic Transfer Switch				
BACK-UP GEN	Rate-MTTR	0,00015	24	1	Back-up generator set				
BEARING (GATE)	Rate-MTTR	4,56E-07	168	1	Bearings of gate fail	Assumed			
BEARING BRACKET	Rate-MTTR	2,279E-07	168	1	Bearing bracket fail	Assumed			
BEARING BRACKETS (WINCH)	Rate-MTTR	0	0	1	Bearing bracket fails				
BEARING CYL	Rate-MTTR	4,56E-07	48	1	Bearing cylinder				
BILGE PUMPS SYS	Rate-MTTR	0,0002	24	1	Bilge pump system (2 No)				
BLIKSEMINSLAG (72)	Rate-MTTR	8,2E-10	72	1	Systemen falen door blikseminslag	Verlies essentiële voorzieningen. Waarbij een bliksembeveiliging klasse LPL II en een overspanningbeveiliging conform NEN-EN-IEC-62305-4 wordt toegepast.			
BOGIE	Rate-MTTR	4,5E-07	168	1	Bogie structure				
BRAKE	Rate-MTTR	3E-06	168	1	Brake				
BRAKE SYSTEM	Rate-MTTR	4,57E-06	24	1	Brake system fails				
BRAND (72)	Rate-MTTR	1,6E-07	72	1	Systemen falen door brand	Aanname: Het aantal industriële gebouwen in Nederland bedraagt circa 700.000. Het aantal branden per jaar met schade in een of andere vorm in industriële gebouwen bedraagt circa 1000 [bron: CBS statline]			

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
						Dit laatste getal is conservatief aangezien alle vormen van schades worden meegenomen. Daarnaast zijn de gegevens afkomstig uit de industriële sector, waaronder ook gebouwen met een hoog brandrisico zoals in de agrarische en petrochemische sector. De kans op een op brand wordt daarmee van 1,42 E-3 / jaar of eenmaal per 700 jaar. (1,6308 E-7 / uur)			
BREAK VALVE	Rate-MTTR	1,7E-06	1	1	Break valve fails	Aangenomen is dat een remventiel vergelijkbaar is met een regelklep. TAW leidraad kunstwerken Mei 2003 Tabel B3.9 Regelklep verstopt = 1,7E-06			
BREAKER VALVE	Rate-MTTR	3,3E-07	168	1	Breaker valve cylinder				
BREAKING RESISTANCE	Rate-MTTR	1E-06	24	1	Breaking resistance fails				
BUSBAR	Rate-MTTR	4,5E-08	16	1	Busbar fails	TAW Leidraad kunstwerken 2003 Tabel B3.7. Rail aan aarde = 4,5E-08			
CABLE DRUM	Rate-MTTR	4,5E-07	730	1	Cable drum				
CABLES	Rate-MTTR	2,25E-07	730	1					

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
CARDAN SUSPENSION	Rate-MTTR	4,4E-07	730	1	Cardan suspension				
CIRCUIT BREAKER	Rate-MTTR	5E-07	16	1	Circuit breaker fails	TAW Leidraad kunstwerken 2003 Tabel B3.7. Schakelaar (vermogens) opent spontaan = 5,0.10-7[/h]			
CIVIL INLET	Rate-MTTR	2,2E-07	8766	1					
CONTR CHECK VALVE	Rate-MTTR	3,3E-07	24	1	Controlled check valve				
CONTROL / SIGNAL CABLES	Rate-MTTR	2E-05	48	1	Control / signal cables fail	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 Kabel (in goot) breekt = 1,2.10-7[/km/h] 2 km kabel = 2,4 10-7[/km/h]			
CONTROL CAB	Rate-MTTR	2,5E-07	24	1	Control and signal cables				
CONTROLLED N.R.V.	Rate-MTTR	4,4E-06	1	1	Controlled non return valve fails	Aangenomen is dat een voorgestuurde terugslagklep vergelijkbaar is met een druk belaste veiligheidsklep. CPR-12E. Blz. 6.55 Safety valve, pilot operated, Spurious opening = 4,4E-06			
CORRECTION	Fixed	0	0	1					
COUNTERWEIGHT	Rate-MTTR	2,2E-07	4382	1					
CPU	Rate-MTTR	5,1E-05	48	1	CPU fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 CPU valt uit = 5,1.10-5[/h]			

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
CYLINDER	Rate-MTTR	1E-07	168	1					
DISCONNECTING SWITCH	Rate-MTTR	5E-07	16	1	Disconnecting switch fails	TAW Leidraad kunstwerken 2003 Tabel B3.7. Schakelaar (vermogens) opent spontaan = 5,0.10-7[/h]			
EL LV CABLES	Rate-MTTR	1,2E-07	16	1	Electric LV Cables				
ELECTRIC MOTOR	Rate-MTTR	3E-05	168	1	Electric motor (system)				
ELECTRIC MOTOR (WINCH)	Rate-MTTR	5E-06	24	1	electric motor fails				
ELECTRONICS CYLINDER	Rate-MTTR	4E-06	168	1	Electronics cylinder				
ELECTRONICS HYDR UNIT	Rate-MTTR	9E-06	168	1	Electronics hydraulic unit				
ENCODER	Rate-MTTR	4E-06	24	1	Encoder				
EQUIV OP LEV	Rate-MTTR	7,724E-06	191,75	1	Operation system leveling system				
EQUIV OP MECH DOOR	Rate-MTTR	1,261E-05	355	1	Equivalent operation mechanisn door				
EQUIVALENT DOOR LOCK	Rate-MTTR	0,0002047	164,4	1					
FLOW DIVIDER	Rate-MTTR	4E-06	48	1	Rotating Flow divider fails				
GEAR RACK	Rate-MTTR	1,3E-06	168	1	Gear Rack				
GEARBOX	Rate-MTTR	1,3E-06	1460	1	Gearbox				
GEARBOX WINCH	Rate-MTTR	3E-09	24	1	Gearbox fails				
GUIDANCE SYSTEM	Rate-MTTR	4,4E-07	168	1	Guidande system				
HIGH WATER ALARM	Rate-MTTR	1E-05	24	1	High water alarm				
HV BOARD	Rate-MTTR	2,5E-06	168	1	High Voltage Board				
HYDR OPERATION MECHANISM	Rate-MTTR	3,034E-05	180,444	1					

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
HYDR. HOSE	Rate-MTTR	4E-06	48	1	Hydraulic hose fails	TAW table 3.9 Hydraulic hose, heavily loaded			
HYDRAULIC CYLINDER	Rate-MTTR	1E-07	168	1	Hydraulic cilinder fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.9 cilinder = 1,0.10-7 [/h]			
HYDRAULIC PUMP	Rate-MTTR	8,5E-05	168	1	Hydraulic pump (complete)				
HYDRAULIC PUMP (CPR)	Rate-MTTR	4E-06	48	1	Hydraulic pump fails	CPR-12E. Blz. 6.55 Pumps. Reciprocating, Motor driven. Fails to run = 4.3E-05			
HYDRAULIC TANK	Rate-MTTR	5E-07	48	1	Hydraulic tank fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.9 Tank. lekkage = 5,0.10-7 [/h]			
HYDRAULIC VALVE	Rate-MTTR	2,6E-07	24	1	Hydraulic valve (solenoid) fails	CPR-12E. Blz. 6.55 valves - Solenoid - failt to operate 2.6E-07			
I/O MODULE	Rate-MTTR	9E-06	48	1	I/O fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 I/O kaart valt uit = 9,0.10-6[/h]			
INST HYDRAULICS	Rate-MTTR	1,6E-06	24	1	Intrumentation hydraulic system				
INTERMEDIATE GEAR	Rate-MTTR	4,5E-07	730	1					
JUNCTION BOX	Rate-MTTR	1,14E-06	24	1	Junction box fails				
KEY MOUSE	Rate-MTTR	1E-05	8	1	Keyboard and mouse				
LEVEL DETECT	Rate-MTTR	1,1E-05	24	1	Level detection				

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
LEVEL DETECTION	Rate-MTTR	1,1E-05	8	1	Level detection	TAW Leidraad kunstwerken. Tabel B3.12. Niveaumeter systeem = 1,1.10-5[/h] faalfrequentie is opgebouwd uit het falen van: Druksensor, of meetversterker of Solid state divice.			
LEVEL SENSOR	Rate-MTTR	3,2E-06	24	1	Level sensor (system)				
LIFTING TOWER	Rate-MTTR	2,2E-07	730	1					
LIMIT SWITCH	Rate-MTTR	7,2E-07	24	1					
LOCAL CONTROL PAN	Rate-MTTR	0,0001	24	1	Local control panels				
LOKALE BEDIENING	Dormant	1,142E-06	8	8760	Lokale bediening van de schuiven faalt.	Expert judgement. faalfrequentie lokale bediening = eens per 100 jaar 1,142E-06			
LV BOARD	Rate-MTTR	8E-06	24	1	Low Voltage Board				
LV CABLES	Rate-MTTR	1,2E-07	16	1	Low voltage cabels fail	TAW Leidraad kunstwerken 2003 Tabel B3.7. Kabel (in goot) breekt = 1,2E-07 [/km/h]			
M.O. HYDRAILIC PUMP	Rate-MTTR	2,283E-06	48	1	Manual operation of hydraulic pump fails	Expert judgement Terugslagklep faalt eens per 50 jaar. 2.283E-06 per uur			
MANUAL OP GATE	Fixed	1E-06	0	1	Manual operation gate. The gate must be able to move on its own weight.				
MANUAL OPERATION	Fixed	1,15E-06	24	1	Manual operation				

Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
MEMORY MODULE	Rate-MTTR	1E-05	48	1	Memory module fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 Geheugen module valt uit = 1,0.10-5[/h]			
MOMENT BEVEILIGING	Rate-MTTR	7,2E-07	16	1	Moment beveiliging faalt	Aangenomen wordt dat een moment beveiliging vergelijkbaar is met een eindsachakelaar. CPR-12E. Blz. 6.56 Electronic limit switch, Spurious operation = 7,2E-07			
MOTOR HYDRAULIC PUMP	Rate-MTTR	9,51E-06	48	1	Motor of hydraulic pomp fails	OREDA Taxonomy no: 2.2.2.4 Electric Equipment - Electric motors - Pump - Cooling systems critical failures: 9.51E-6 /H			
MOTOR PROTECTION	Rate-MTTR	3E-07	48	1	Motor protection fails	TAW 2003+RHK			
MOTOR STARTER	Rate-MTTR	5E-06	48	1	Motor starter fails	Emotron			
OIL LEVEL SENSOR	Rate-MTTR	5,2E-07	24	1	Oil level sensor				
OIL RESERVOIR	Rate-MTTR	1,1E-09	730	1	Oil reservoir				
OPEN GEAR DRIVE	Rate-MTTR	9,9E-10	24	1	Open gear drive fails				
OPERATION MECH GENERIC	Rate-MTTR	8,75E-06	48	1	Generic for operation mechanism				
PINION GEAR	Rate-MTTR	4,5E-07	730	1	Pinion Gear				
PLC SYSTEM	Rate-MTTR	1E-05	24	1	PLC system (complete)				
POSITION DETECTION	Rate-MTTR	4E-06	48	1	Absoluut position detection				

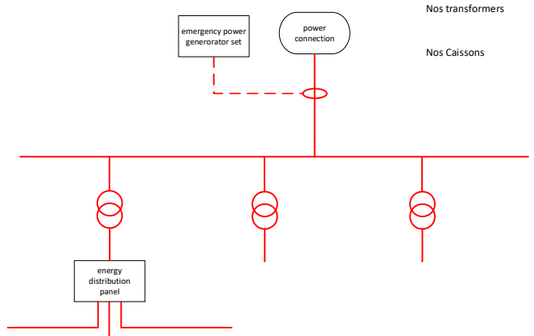
Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
POWER GRID	Rate-MTTR	3,8E-05	1,5	1	Powergrid				
POWER PLANT	Rate-MTTR	3,8E-05	1,5	1	Power plant	Netbeheer nederland			
POWER SUP. PLC	Rate-MTTR	2,9E-05	48	1	Power supply PLC fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 Voeding valt uit = 2,9.10-5[/h]			
PRES SENSOR	Rate-MTTR	6,6E-07	24	1	Pressure sensor				
PRESSURE SAFETY	Rate-MTTR	8,6E-07	24	1	Pressure safety of hydraulic unit fails	Aangenomen wordt dat een drukbeveiliging vergelijkbaar is met een veer belaste veiligheidsklep. CPR-12E. Blz. 6.55 Safety valve, spring operated, Spurious opening = 8,6E-07			
RELIEVE VALVE	Rate-MTTR	9,22E-07	1	1	Pressure relieve valve				
RISTRICTOR VALVE	Rate-MTTR	6E-06	1	1	Ristrictor valve fails				
ROTATING FLOW DIVIDER	Rate-MTTR	4E-06	48	1	Rotating Flow divider fails				
SCADA	Rate-MTTR	1E-05	24	1	SCADA system				
SCADA SERVER	Rate-MTTR	8,33E-06	24	1					
SCHEAVE CONSTRUCTION	Rate-MTTR	1E-07	730	1					
SCHUIFAFSLUITER	Dormant	1,4E-07	48	8760	Schuifafsluiter faalt (sluit niet af)	OREDA 2002. Taxonomy: 4.3.5. Valves gate. External leakage = 0.14E-06			
SCREEN	Rate-MTTR	2E-05	8	1	Screen				
SOFTWARE	Rate-MTTR	1E-06	48	1	Software fails	TOPAAS			

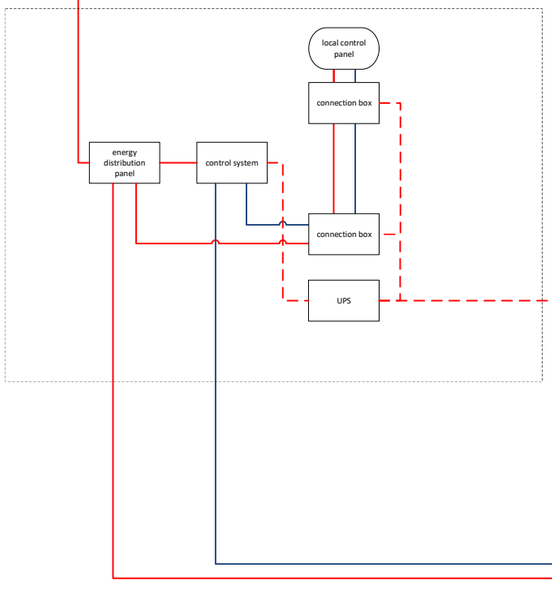
Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
						Softwarefalen = 1,0E-06. score -1/0/0/0/-0,5/-0,33/-0,33/0/0/-0,5/-1/1/0/-1/-0,33; Software is niet complex en wordt volledig getest. de kans op falen door de software is daardoor nihil			
SOLENOID	Rate-MTTR	8,9E-07	168	1	Solenoid				
SPINDLE LIMIT SWITCH	Rate-MTTR	3E-05	24	1					
STEEL CABLE	Rate-MTTR	1,2E-09	24	1	Steel cable fails				
STEEL HYDRAULIC PIPE	Rate-MTTR	5,7E-08	48	1	Stainless steel pipes fail	TAW leidraad kunstwerken Mei 2003 Tabel B3.9 Slang licht belast = 4,0.10-6 [/h]			
STEEL PIPES	Rate-MTTR	4E-06	168	1	Stainless steel pipes fail				
STEEL ST GATE	Rate-MTTR	2,2E-07	4383	1					
STEEL STRUCTURE	Rate-MTTR	2,279E-07	4380	1	Steel structure fails	Assumed			
THERMAL PROTECTION	Rate-MTTR	3E-07	16	1	Thermal protection fails	TAW leidraad kunstwerken Mei 2003 Tabel B3.7 Thermische beveiliging. Voortijdig open = 3,0.10-7 [/h]			
TORSION SHAFT	Rate-MTTR	6,6E-10	24	1	Torsion shaft fails				
TRANSFORMER	Rate-MTTR	1E-06	72	1	Transformer fails	TAW Leidraad kunstwerken 2003 Tabel B3.12.			

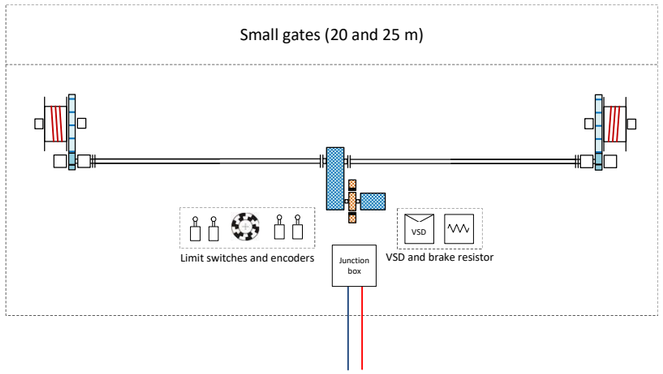
Model Name	Model type	Failure rate	MTTR	Test interval	Description	Source	Reference	Remarks	Date
						Transformator = 1,0.10-6[/h]			
TRANSFORMER GEEN BRON	Rate-MTTR	1E-06	168	1	Transformer				
VFD	Rate-MTTR	5E-06	48	1	Variable speed drive				
VSD (WINCH)	Rate-MTTR	5E-06	24	1	Variable speed drive fails				
WIRE DRUM	Rate-MTTR	8,25E-10	24	1	Wire drum fails				

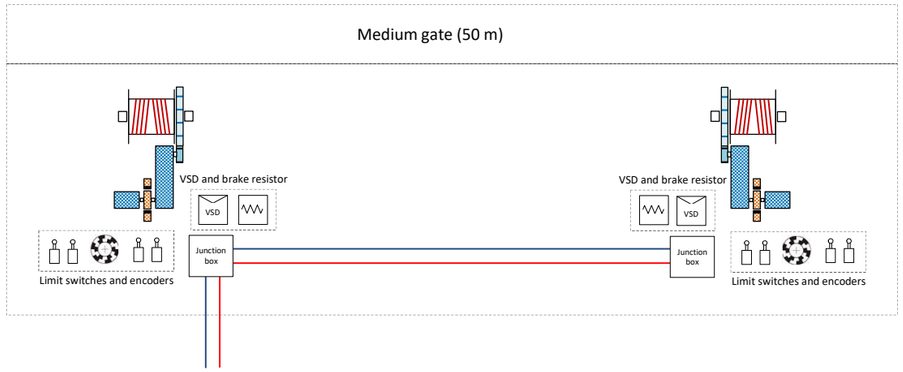
A13 PRINCIPLE OF ELECTROTECHNICAL OPERATING MECHANISM

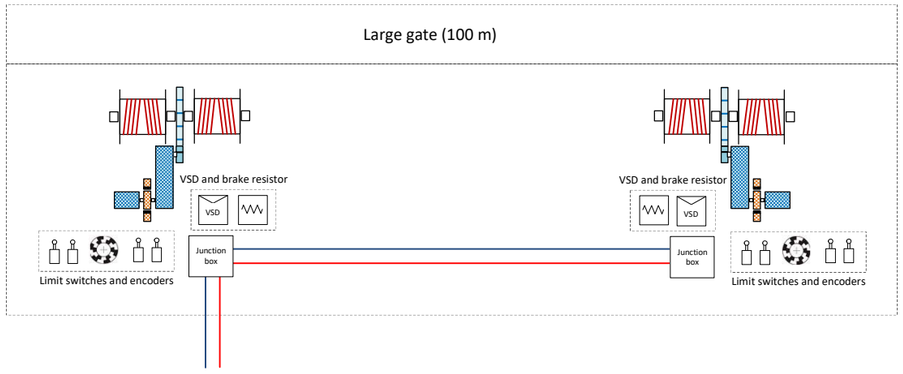
	Dam					
	1 km	5 km	10 km	15 km	20 km	25 km
Nos transformers	1	3	7	10	14	17
Nos Caissons	Transformer					
	14					





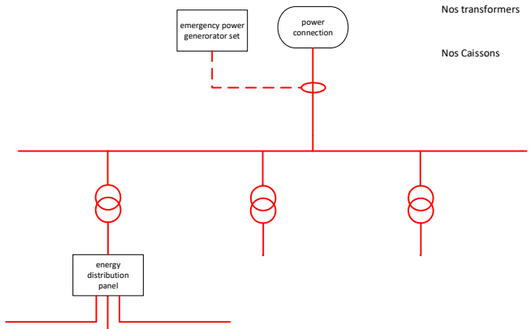


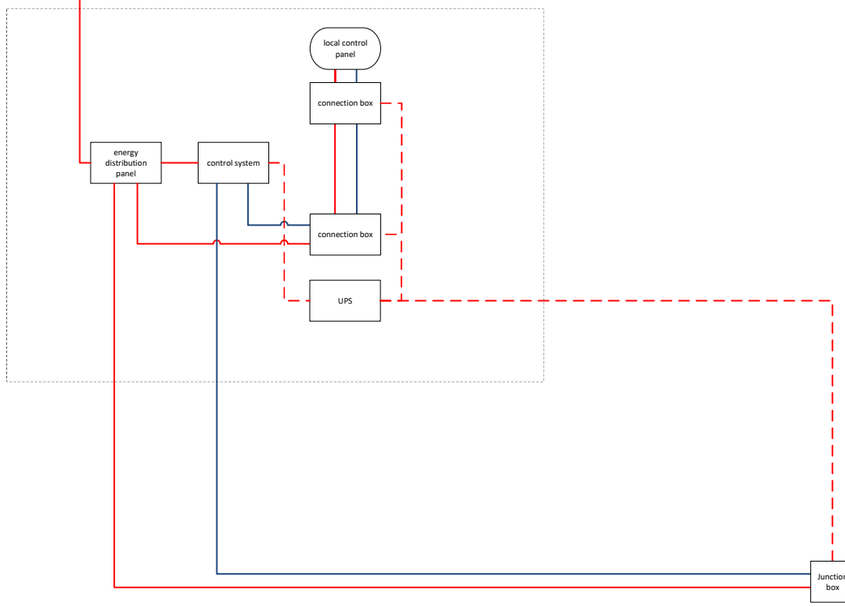


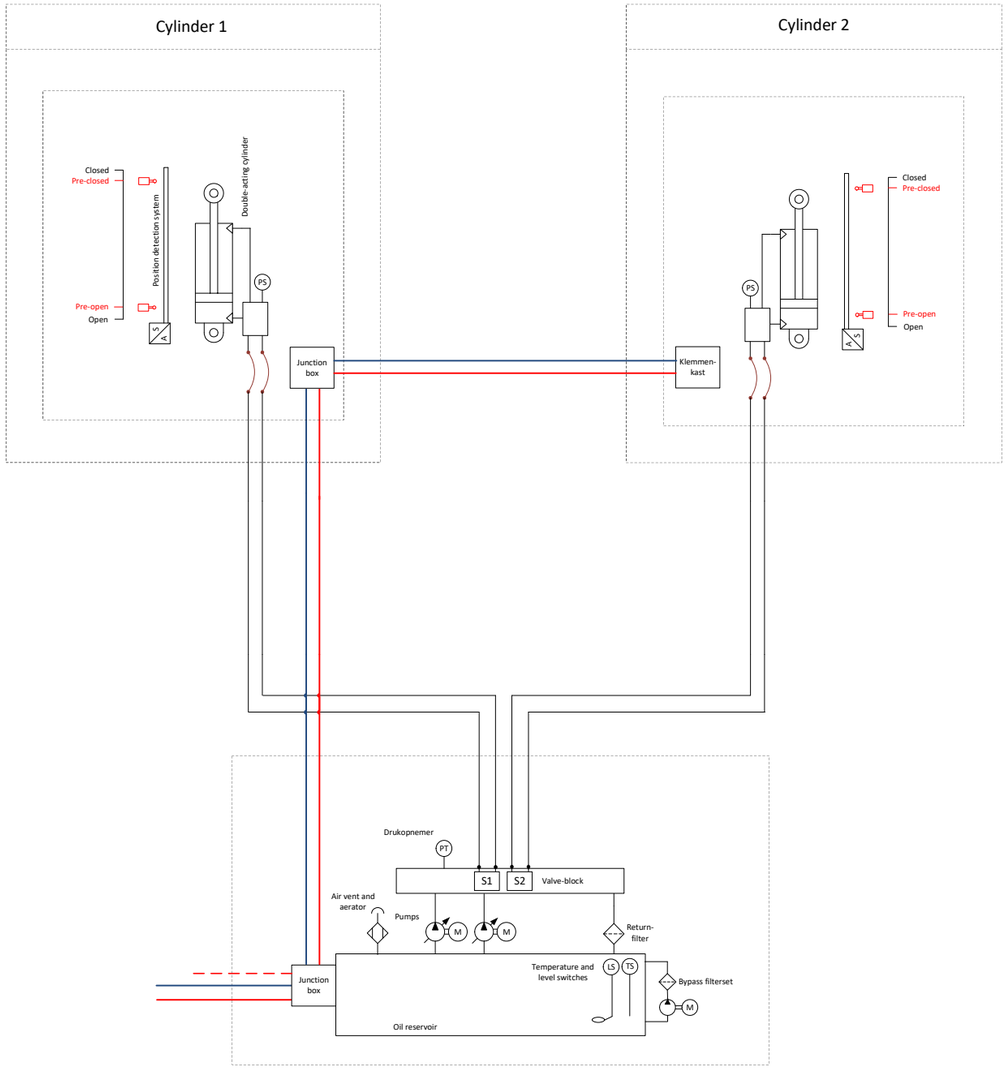


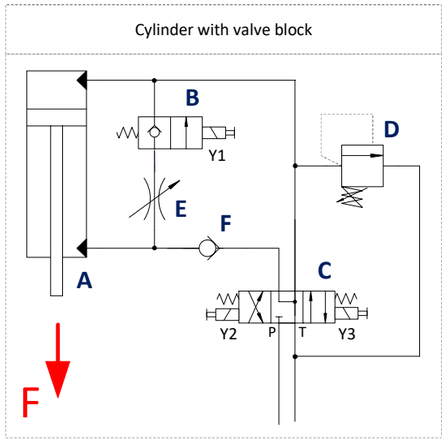
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Status:		Versie:	Tekeningnr.:

	Dam					
	1 km	5 km	10 km	15 km	20 km	25 km
Nos transformers	1	3	7	10	14	17
Nos Caissons	Transformer 14					



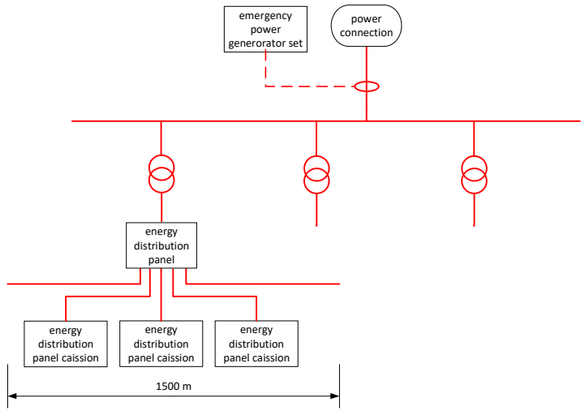


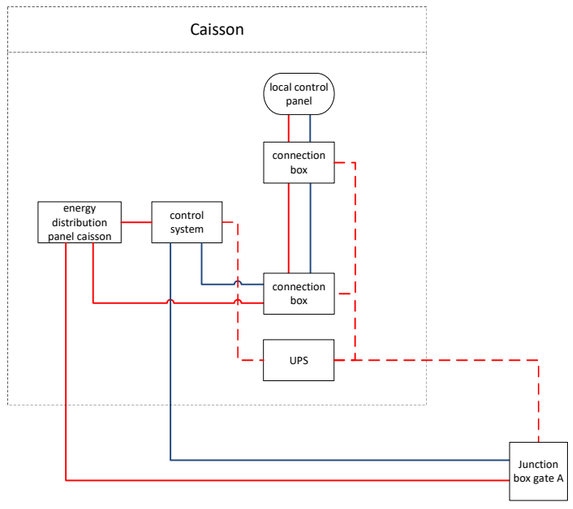




A: Dubble action cylinder
B: Safety and differential valve
C: Direction valve
D: Pressure relieve valve
E: Restrictor valve
F: Non-return valve

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REPORT

Volume 3: Technical Report

Design review of permanent structures Kalpasar Dam and design of spillway gates

Client:	India Ministry of Earth Sciences, National Centre for Coastal Research (NCCR)
Reference:	BI5332-101-100-RHD-ZZ-XX-RP-Z-0002
Status:	Final/01
Date:	3 February 2023

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Document title: Volume 3: Technical Report

Subtitle: Design review of permanent structures Kalpasar Dam and design of spillway gates
Reference: BI5332-101-100-RHD-ZZ-XX-RP-Z-0002
Status: 01/Final
Date: 3 February 2023
Project name: Kalpasar dam project
Project number: BI5332-101-100
Author(s): Robert Jacobs, Geert Smits

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Checked by:	Shaun Chamberlain
Date / initials:	1 February 2023
Approved by:	Satyanand Kunta
Date / initials:	3 February 2023

Classification
Project related

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1 General

1.1 Introduction

Identified first in the 1970's for its tidal power resource and having been considered in different configurations over the years, at present the Kalpasar Dam aims to provide a reliable source of fresh water for the drought-stricken state of Gujarat (Saurashtra in particular) by means of a 60 km dam between Bhavnagar and Aladar of which the centre 30km concerns a closure dam in the Gulf of Khambhat (Figure 1-1). A reliable source of fresh water is establsihed once the closure dam is constructed and the transition from sea water reservoir to fresh water reservoir has occurred. From earlier studies it was found that the construction of the closure dam and the saltwater transition will require 10 to 15 years before fresh water is available, pending further analysis beyond the present scope of this assignment.

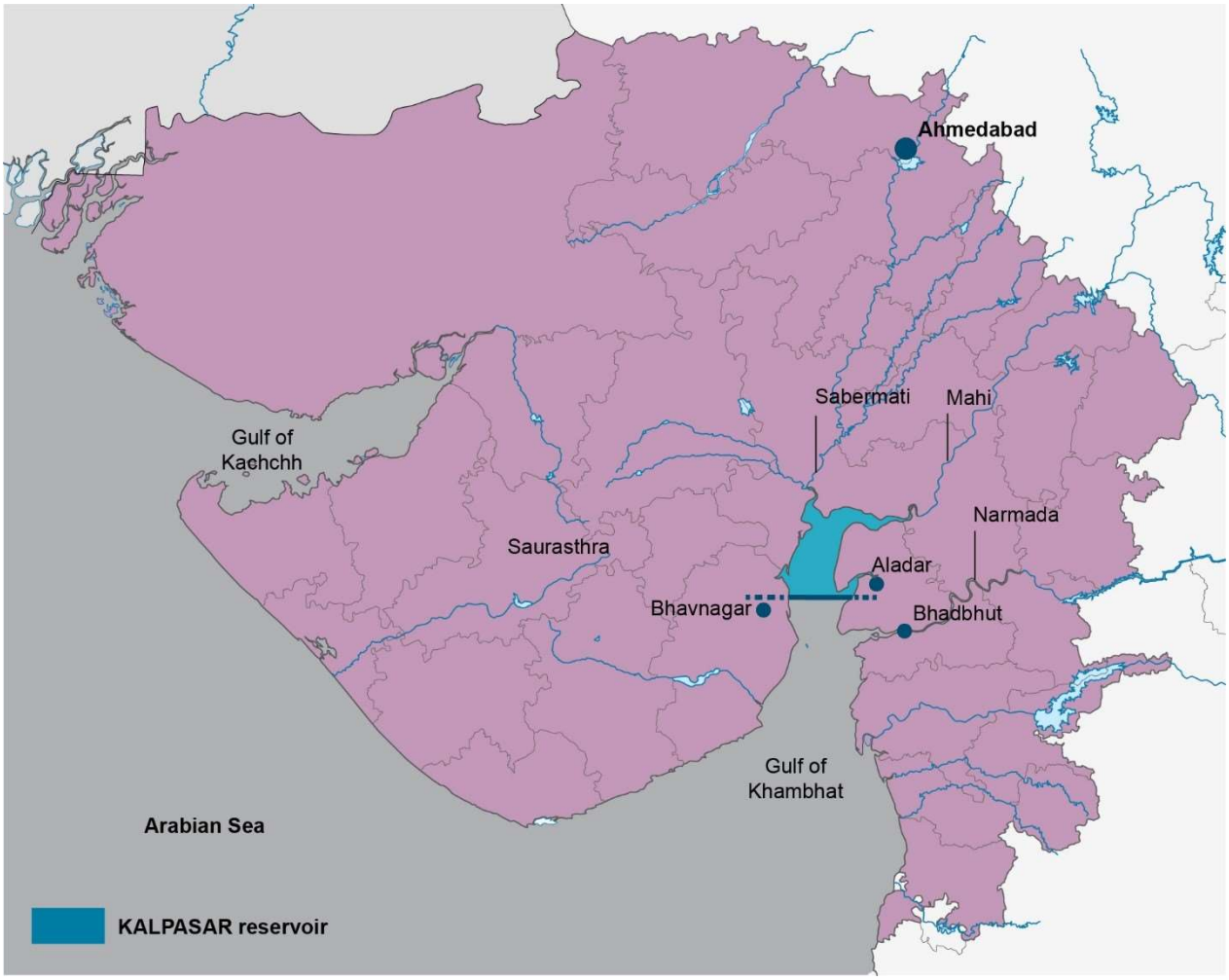


Figure 1-1: Selected alignment of the Kalpasar Dam in the Gulf of Khambhat

In its present form, the Kalpasar project includes the 60 km dam connecting the Bhavnagar and Bharuch districts and a road/rail corridor, a freshwater reservoir of 7.800 million m^3 and a spillway to divert floods from the three major rivers flowing into the freshwater reservoir (Sabarmati, Mahi and Dhadhar). Once completed, a diversion from the Bhadbhut Barrage in the Narmada by means of a canal towards the

freshwater reservoir is also considered by Narmada Water Resources, Water Supply and Kalpasar Department (NWRWS).

The Kalpasar Dam is a huge undertaking considering the sheer volume of work and associated cost alone. Considering that during construction, (i) site conditions of more than 25 m water depth on average with poor soil conditions, (ii) a 4 m to over 10 m tidal range at Bhavnagar; and (iii) monsoon seasons that take out 3 months of working time each year must be accounted for amongst others, makes the project even more challenging and costly. A delay in its completion – if construction risks are not appropriately mitigated – is likely to happen. Given the anticipated high cost of construction, the risk of delay before fresh water becomes finally available results in a viability gap that is difficult to mitigate, if at all. As such, Kalpasar Dam is an unprecedented project.

The construction of the Kalpasar Dam consists of two main stages: (i) the construction of the closure dam and (ii) the upgrading of the closure dam to the final (permanent) dam. The present feasibility study considers the feasibility of the closure dam only, seeking to identify an envelope of closure strategies with optimised construction risks. This is the first and probably most important step in de-risking Kalpasar project. After all, once the closure dam is erected, the construction risk associated with upgrading it to its desired final form is dramatically lower, although still significant for its size.

1.2 Overview of scope of assignment

The scope of the present assignment consists of two aspects, namely:

1. Design of Closure Methodology for Kalpasar Dam, comprising:
    a. Review of existing closure methodology reports and study various options for closure of Kalpasar Dam; and
    b. Arrive at optimal closure methodology and design the same.
2. Design of gate and hydraulic mechanical system.

This scope has been executed and documented as described in the following sub-chapter.

1.3 Guidance for the reader

The execution of the study is documented in a Detailed Project Report with four supplementary Technical Volumes as annexures, structured as follows:

- Detailed Project Report (DPR)
    - The Detailed Project Report (DPR) takes the results from the technical report on the Kalpasar closure dam feasibility (Volume 2) to assess and evaluate risks to account for when considering the alternatives identified. Risks addressed are amongst others volume risks, price risks, time related risks and the failure mode analysis as (key) technical risks and mitigated in their respective designs. The DPR includes an executive summary, key issues and risks to consider and recommendations on the way forward.
- Volume 1: Technical Report on Hydrodynamic Modelling of Kalpasar Dam Closure
    - Volume 1 introduces the closure methods and strategies, and describes the hydraulic analysis based on thousands of simulations with the one-dimensional model hydraulic model, conducted to determine current velocities occurring during the closure for different closure options.
- Volume 2: Technical Report on Kalpasar Closure Dam Feasibility
    - From a thorough analysis of site conditions, a design basis for Kalpasar closure dam is determined to provide an unequivocal and solid basis for (i) development of distinct and

feasible alternatives and (ii) risks to account for in the feasibility analysis presented in the Detailed Project Report. Gradual and sudden closure components are designed according to the design basis and used as building blocks allowing for evaluation of different closure scenarios in a construction model that provides insight into cost and duration of construction. From the evaluation of scenarios, the distinct closure strategies are derived and evaluated for input to the feasibility analysis. The construction model used results in a theoretical design that is subsequently translated into a practical design for (i) illustrative purposes and (ii) to serve as a basis for the Bill of Quantities and Cost Estimates in Volume 4.

- Volume 3: Technical Report on Design Review of Permanent Structures Kalpasar Dam and Design of Spillway Gates **(THIS REPORT)**
  - Volume 3 contains the identification and assessment of options for the spillway gates, concluding in a conceptual design of the preferred option, a cost estimate and an operations and maintenance plan. In addition, a review of the Kalpasar Dam structures in their permanent form, including the spillway, is provided as additional feedback on the designs by other consultants provided by the client.
- Volume 4: Technical Report on Cost Estimate for Kalpasar Closure Dam
  - Volume 4 contains specifications, a bill of quantities and an associated cost estimate for the Kalpasar closure dam elements.

This Volume 3 Report is further broken structured to provide the necessary information as follows. A General introduction is provided in **Chapter 1. Chapter 2** aims to review the basic design of the permanent structure of the Kalpasar Dam and the spillway as provided by the client. In **Chapter 3**, alternatives for the spillway gate type are identified, compared, and a multi-criteria analysis is carried out to determine a most favourable gate type option. The gate operational mechanism is then considered and selected. **Chapter 4** considers design, fabrication, maintenance, functionality, aesthetic and cost aspects of the preferred spillway gate, and concludes with a conceptual design of the preferred gate, complete with estimated quantities and a cost estimate. **Chapter 5** provides an operation and maintenance plan for the spillway gates, including replacement frequencies, cost estimation, operational parameters and maintenance activities required. Risks and future considerations related to the spillway gates are outlined in **Chapter 6**. **Chapter 7** highlights the conclusions of the technical report related to the spillway gates. And finally **Chapter 8** provides a list of references used in the report.

2 Review of permanent Kalpasar Dam structures

The permanent structure of the Kalpasar Dam is a 60km long dam across the Gulf of Khambat in India for establishing a huge fresh water coastal reservoir. A road link will also be set up over the dam, greatly reducing the distance between Saurashtra and South Gujarat.

For the preferred closure methodology as determined in Volume 1, a technical conceptual design will be prepared. Based on the subsoil characteristics, bathymetry and the hydraulic loads, approximately 5 to maximum 10 typical cross- sections will be selected along the dam. For these cross-sections, a design of the dam will be prepared. This design includes the foundation, core, slope protection and bed protection. An important aspect to consider in the design, is the difference between the (temporary) construction design and the design in operational conditions once the permanent dam has been constructed. There will be a period (est. 2 to 5 years) where the closure dam must be able to withstand load conditions before the final dam is constructed. The dominant design condition will be a combination of water level and wave height.

The designed cross-sections will be interpolated into a technical 3D model (Civil3D), in order to provide material quantities for the different components of the dam. The Bill of Quantities is input for the cost estimates.

The protection of the closure dam (the temporarily structures, consisting of a combination of rock dams and caissons) and the protection of the reclamation area behind the protection structure/breakwater, is done with a permanent breakwater-type structure on the seaside of the closure dam.

In this chapter, the concept design regarding the seaside of the breakwater structure at various bed levels considering different design wave heights will be described. This design has been tested in a wave flume at facilities of CWPRS [1].

2.1 Design principles

In this phase, all activities help define a concept design for the most appropriate dike design, including:

- Initial data gathering
- Additional data collection
- Compilation of design criteria
- Concept design of dike cross sections

The concept design of the preliminary cross-sections of the closure dam covers the following:

- The dike cross-sections (levels, slope gradients)
- The main armour on front, crest and rear side of the dike
- The main filter layers
- The toe structures
- The monolith elements (main dimensions), if applicable

This is not a list of separate and distinct design steps; there is a clear interaction between these different elements. In the design process the way the different elements inter-relate is continuously considered, both in the hydraulic and the geotechnical design of the structure.

The following parameters will be considered for the conceptual design of the dike:

- Height:
    - Wave overtopping

- Subsoil settlement
- Stability:
  - Stability armour layers seaside
    - Stability of concrete armour units
    - Stability of rock armour
    - Rock toe stability
  - Overtopping resistance crown and inside slope

The reservoir side of the dam is at the moment not considered due to lack of information regarding wave heights at the reservoir side of the dam.

2.2 Design conditions

For the concept design of the dike, the following parameters are needed:

- The design significant wave height, Hs (either Hm0 or $H_{1/3}$)
- The wave period, Tp
- Bathymetry
- Design water level
- Soil characteristics

Cross-sections
Furthermore, 5 to 10 cross-sections along the dike should be determined at which the conditions (hydrodynamic and soil) are preferably known to come up with a concept design for these cross-sections.

The dike runs from Bhavnagar to Dahej for about 33km, so a cross-section roughly every 5km gives 7 cross-sections along the dike. In previous studies, IITM report, cross-sections are analysed with different depths, namely 6 different cross sections of 24m, 29m, 34m, 39m, 44m and 49m high, and corresponding reduced levels of the seabed of -5m, -10m, -15m, -20m, -25m, and -30m. The geotechnical stability for these cross sections has been determined in the report of IITM.

In the CWPRS report cross-sections have been tested in a wave flume to check the stability of the armour layer and to determine the amount of wave overtopping. The cross sections were tested according to the seabed level in front of the toe of the structure, as the loads on the structure are dependent on the design wave height and on the water depth in front of the structure. The cross sections with a seabed level of -5m to -30m have a relatively large water depth and the design wave is not depth limited in these areas of the dam. Therefore, the design of the armour layer in these cross sections do not change. In shallower water depths, the design wave height becomes depth limited and the design wave height will reduce. The following cross sections with bed levels will be considered in the concept design:

-25.0 m, -5.0 m, 0.0 m, +2.0m and +5.0m.

The design wave height
Wave studies have not been carried to determine the design conditions, therefore the design wave height is taken from the report of the Wave Flume Studies of CWPRS [1].

The design wave height in this report was considered to be H_s = 8.1 m.

The corresponding wave period for the design wave height, assuming T_p, in the CWPRS report is 12 sec.
Design water level

The design water level is taken from the CWPRS report. The design water level in this report was considered to be DWL = +8.765 m w. r. to MSL.

Other water levels to be considered are:

HAT = +5.3 m w.r. to MSL
MSL = +0.0 m w.r. to MSL
LLWL = -4.8 m w.r. to MSL

2.3 Materials

To keep the costs of construction as low as possible the materials should be taken from nearby locations.

Core
The core of the dam can consist of a quarry run for the volume filling. The optimal specification requires an understanding of the performance of the differing types of core materials, key parameters often being the porosity or the risk of damage to the structure through piping or internal erosion. For example, in a harbour breakwater, especially where wave transmission is to be limited, the decision to set the fines cut-off at, say 10kg, 1 kg, or at no cut-off may need consideration of:

- Core porosity and, indirectly, permeability
- Percentage utilisation of quarry yield if materials are coming from a dedicated quarry
- Instability because of clay minerals in the fines
- Shear strength and liquefaction potential

Filter layers
Filter layers are needed to prevent the core material to get washed out during the lifetime of the structure. During the construction period of the structure, the filter layer is more hydraulically stable against wave attack, so it prevents damage to the core material during operational conditions, however if a large storm hits the structure during construction some damage is to be expected.

Armour layers
The armour layers on the seaside of the dam must consist of large elements to be hydraulically stable against the wave attack. Also, on the basin side of the dam the waves can become quite significant due to the long fetch. Whether or not to use concrete elements for the armour layers depends on the design conditions, availability of large rocks, ease of installation, etc.

2.4 Conceptual design cross sections

The following parameters of the structure are checked during the conceptual design phase in this report.

- Stability armour layers seaside
    - Stability of concrete armour units
    - Stability of rock armour
    - Rock toe stability
- Wave overtopping
- Overtopping resistance crown and inside slope

2.4.1 Stability armour layers seaside

2.4.1.1 Stability of concrete armour units

The protection structure/breakwater of the Kalpasar Dam is proposed from either side of the bank, extending up to -25m depth contour w. r. to MSL. A decision to design the rubble mound protection structure/breakwater with Accropode II as primary armour was made in a Kalpasar project meeting of 12 November 2022. That decision post-dated the assessment presented below, which also included consideration of XblockPlus units.

The stability of the armour layer is determined according to the Hudson formula. The required volume for the Accropode II units or the XblocPlus units is calculated with this formula, according to:

$$V(H_s, s) = \left(\frac{H_s}{\Delta}\right)^3 \cdot \frac{1}{K_D(s)\cot\alpha}$$

Where,

H_s = significant wave height (m)
$\Delta = (\rho_c - \rho_w)/\rho_w$ = relative buoyant density
cot α = slope of armour
$K_D(s)$ = stability coefficient (depending on the seabed slope, s)

The stability coefficient (K_D) depends on the slope of the seabed and on the type of unit to be used. The K_D values are for non-breaking waves and based on safe engineering practice.

The Accropode II units have a maximum K_D value of 16 (with a seabed slope of 1% and lower). The K_D value of XblocPlus is 12 (DMC, the designers of the XblocPlus block, have chosen a lower K_D value to have more safety in design, and so that fewer units have to be produced and placed). The overall concrete consumption of an armour layer with the XblocPlus unit is still lower than an armour layer with the Accropode II unit or on par with the Xbloc unit despite the lower K_D value.

Cross-sections with bed levels of -5.0 m to -25.0 m
At cross-sections with a seabed level of -5 to -25m CD, the design wave height is 8.1m. At shallower parts of the dam the design wave height is depth limited.

With a design wave height of 8.1m and an armour slope of 1:1.33, the following unit sizes are calculated:

Accropode II : Theoretical volume: 11.8 m3 (with a seabed slope of 2.5%).
XblocPlus: Theoretical volume: 14.1 m3.

CWPRS have conducted physical model tests with the Accropode II units (14m3) with bed levels of -5m CD and -25m CD. These breakwater sections were found to be hydraulically stable under the design wave conditions.

Cross-sections with bed levels of +0.0 m to -5.0 m
At cross sections with a seabed level of +0.0 m to -5.0 m CD, the design wave height can be depth limited (at +0.0m (MSL)), however in deeper parts with a seabed level of -5.0m the design wave of 8.1 can occur.

Therefore, the block sizes are calculated with a design wave height of 8.1m and an armour slope of 1:1.33. The following sizes are calculated:

Accropode II : Theoretical volume: 11.8 m3 (with a seabed slope of 2.5%).
XblocPlus : Theoretical volume: 14.1 m3.
CWPRS have conducted physical model tests with the Accropode II units (14m3) with bed levels of +0.0m CD and -5m CD. These breakwater sections were found to be hydraulically stable under the design wave conditions.

Cross-sections with bed levels of +2.0 m to +0.0 m
At cross sections with a seabed level of +2.0 m to +0.0 m CD, the design wave height is depth limited. The design water level is +8.765 m. So, the design water depth at +0.0m is 8.675m. The resulting significant wave height which could occur at a seabed level of +0.0m is: 8.675 * 0.78 = 6.8m.

With a design wave height of 6.8 m and an armour slope of 1:1.33, the following unit sizes are calculated:

Accropode II : Theoretical volume: 6.1 m3 (with a seabed slope of 1%).
XblocPlus : Theoretical volume: 8.3 m3.

CWPRS have conducted physical model tests with Accropode II units of 6 m3 at a bed level of +0.0m CD. This breakwater section was found to be hydraulically stable under the design wave conditions (wave height of Hs = 6.8m, wave period of 10 to 12s)

Cross-sections with bed levels of +5.0 m to +2.0 m
At cross sections with a bed level of +5.0 m to +2.0 m CD, the design wave height is depth limited. The design water level is +8.765 m. So, the design water depth at +2.0m is 6.675m. The resulting significant wave height which could occur at a seabed level of +2.0m is: 6.675 * 0.78 = 5.3m.

With a design wave height of 5.3 m and an armour slope of 1:1.33, the following unit sizes are calculated:

Accropode II : Theoretical volume: 2.9 m3 (with a seabed slope of 1%).
XblocPlus : Theoretical volume: 4.0 m3.

CWPRS have conducted physical model tests with Accropode II units of 3 m3 at a bed level of +2.0m CD. This breakwater section was found to be hydraulically stable under the design wave conditions (wave height of Hs = 5.3m, wave period of 10s)

Cross-sections with bed levels from root to +5.0 m
At cross sections with a bed level of +5.0 m and higher, the design wave height is depth limited. The design water level is +8.765 m. So, the design water depth at +5.0m is 3.675m. The resulting significant wave height which could occur at a seabed level of +5.0m is: 3.675 * 0.78 = 2.9m.

With a design wave height of 2.9 m and an armour slope of 1:1.33, the following unit sizes are calculated:

Accropode II : Theoretical volume: 0.47 m3 (with a seabed slope of 1%).
XblocPlus : Theoretical volume: 0.65 m3.

CWPRS have not conducted physical model tests at this bed level and with these armour units.

2.4.1.2 Stability of rock armour

The stability of rock armour can be determined according to the Van de Meer formulae as specified in the Rock Manual (2007, CIRIA/CUR) [2].

The Van der Meer formula for rock armour stability in deep water is as follows:

$$\frac{H_s}{\Delta \cdot d_n 50} = \begin{cases} c_s P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi^{-0.5} & if\ \xi_m < \xi_{cr} \quad [plunging] \\ c_s P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^P \sqrt{\cot \alpha} & if\ \xi_m \geq \xi_{cr} \quad [surging] \end{cases}$$

and

$$\xi_{cr} = \left[\frac{c_p}{c_s} P^{0.13} \sqrt{\tan \alpha}\right]^{\frac{1}{P+0.5}}$$

For shallower parts of the dam, bed levels of +5.0m CD and higher, a rock armour could also be a solution. The calculated rock armour with a design wave height of 2.9 m and an armour slope of 1:1.33 is W_{50} = 6.3 ton.

2.4.1.3 Rock toe stability

The rock toe armour stability is determined with the Van der Meer formula according to the Rock Manual [2]. The design method typically applies to deep water toe (large water depths) in non-depth limited waves. See the formula below.

$$\frac{H_s}{\Delta D_{n50}} = \left[2 + 6.2 \left(\frac{h_t}{h}\right)^{2.7}\right] n_{od}^{0.15}$$

The design typically applies to deep water toe in non-depth limited waves. No reduction for oblique wave attack is considered.

At the edge of applicability of the formula the calculated rock toe at seabed level of -15.5m and top of toe level of -9m with a design wave of 8.1m is $W_{50} \approx 10$ ton. Going to deeper parts the formula is outside of its applicability.

CWPRS has tested rock toe sizes at various seabed levels according to reference design, and these sizes have been found to be stable.

2.4.2 Wave overtopping

Wave overtopping will be determined to check if the crest level and the crest widths are sufficient so that overtopping will remain below acceptable limits under design conditions. The crest and rear slope shall not show excessive damage during the design condition as a result of wave overtopping. This leads to an allowable overtopping of q < 50 l/s/m.

According to the EurOtop formula (2018) the wave overtopping during design conditions is about 20 l/s/m for the sections with wave wall crest level heights of +19.0m CD.

2.4.3 Overtopping resistance crown and rear slope

The resulting wave overtopping determined in the previous paragraph must not lead to any damage to the crown and rear slope. According to Van Gent, [Van Gent, M.R.A; rear-side stability of rubble mound

structures with crest element, 2007], the median rock weight of the rear side slope should be at least 3700 kg.

In the reference design and the design tested by CWPRS the rear side slope armour is 2-4 ton. So, this is slightly below the design value determined by the Van Gent formula. However, the rear side armour slope was found to be stable during model tests performed by CWPRS.

2.5 Typical geometry cross sections

2.5.1.1 Cross sections

The permanent structure of the Kalpasar Dam is drawn in a Civil 3D drawings, cross sections at various locations at various depths can be taken out the Civil3D digital model. The scale of the cross sections shown below differs as the width of the dam is proportional to the height of the dam. The width of the asphalt, above excavated material is the same for each cross section shown below. The following cross section is shown of the dam with a seabed level of -20m CD.

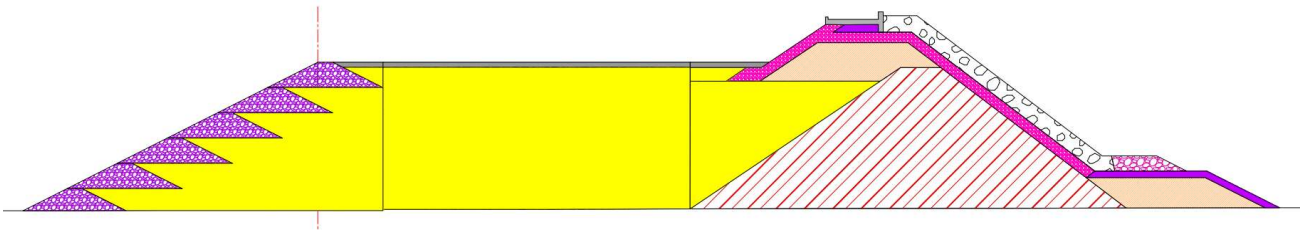


Figure 2-1: Cross section at seabed level of -20m

The following cross section is shown at a seabed level of -5m CD.

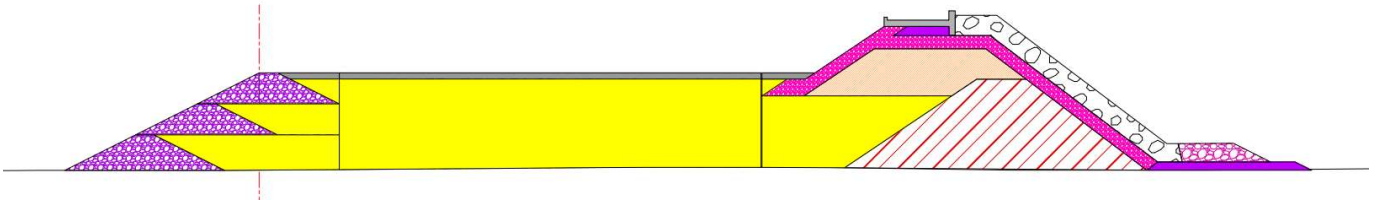


Figure 2-2: Cross section at seabed level of -5m

The following cross section is shown at a seabed level of +0.0 CD.

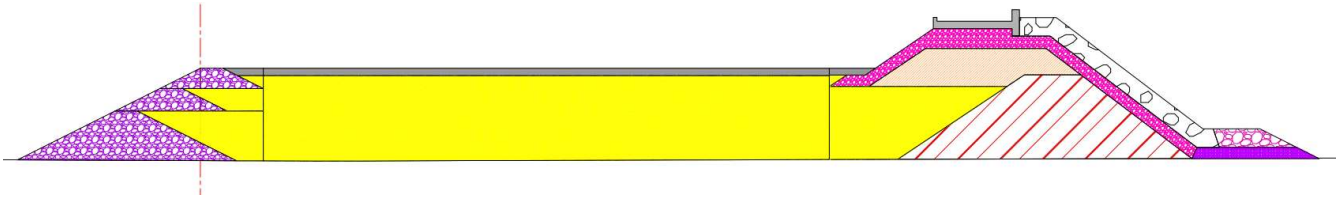


Figure 2-3: Cross section at seabed level of +0.0 CD

2.5.1.2 Volumes

From the Civil3D model of the Kalpasar Dam, the total volumes can be determined needed for construction. A distinction is made between the type of materials. The filter layers underneath the dam, a bedding layer of 1 – 10 kg stone, and between the rock bunds at the reservoir side of the dam and the reclamation material are not considered.

Table 2-1: Material volumes

Item No.	Unit	Material	Total [m3]
1	m3	Volume closure dam	26,781,981
2	m3	Dredged / excavated material	72,129,078
3	m3	Core material (10 - 500 kg)	6,899,828
4	m3	Concrete	529,395
5	m3	Asphalt	3,127,709
6	m3	Rock 200 - 300 kg	70,044
7	m3	Rock 300 - 500 kg	9,922,633
8	m3	Rock 0.3 - 3 ton	10,273
9	m3	Rock 0.5 - 1 ton	1,481,630
10	m3	Rock 1 - 2 ton	712,219
11	m3	Rock 2 - 4 ton	4,994,295
12	m3	Rock 4 - 6 ton	140,560
13	m3	Rock 8 - 10 ton	978,633
14	m3	1.6 ton (0.65m3) XblocPlus or 1.2 ton (0.5m3) Accropode II	20,807
15	m3	9.6 ton (4m3) XblocPlus or 7.2 ton (3m3) Accropode II	36,329
16	m3	19.2 ton (8m3) XblocPlus or 14.4 ton (6m3) Accropode II	499,695
17	m3	33.6 ton (14m3) XblocPlus or 33.6 ton (14m3) Accropode II	4,871,069

2.6 Scour protection at spillway

2.6.1 Context

In the course of execution of the assigment, the client raised concern regarding scour protection at the Kalpasar Dam spillway. This is indeed a valid concern. Hence, notwithstanding the fact that design of scour protection at the spillway is outside of the scope of this assigment, we have nonetheless provided some comments below borne out of our previous experience with such issues, to assist the client in moving forward on the matter. Note that these comments are provided without having seen details of any current spillway scour protection design.

2.6.2 Considerations

- During opening and closing of the spillway gates the bottom of the spillway in front and back area must be protected and stabilized.
- The situation with high discharges and significant head differences over the spillway are critical for the scour protection of these areas.
- The dimensioning of the areas which must be protected depend mostly on the local soil conditions and flow conditions of the water.
- Advanced calculations must be elaborated to determine the criticality of the transition situations (opening and closing situation of the gates). To avoid an unstable hydraulic jump, it is important that the vertical lifting gates will be operated very accurately and slowly, in such a way that the water level in the upstream area is able to follow the changes in the discharge through the spillway in the transition period to and from the flushing regime (to reach equilibrium depth in the downstream area). In contrast, if the vertical lifting gates are lifted too quickly during the opening phase, higher velocities and possibly an unstable hydraulic jump downstream of the spillway.
- There are many ways to protect the bottom areas around the spillway. Frequently used types are made up of one or more layers of loose granular material. In addition to the fully granular scour protection, the following alternative constructions are possible:
    - rocks/stone on a geotextile;
    - block mat;
    - gabions;
    - mattresses;
    - plate-shaped (concrete) structures; and/or
    - underwater concrete (anchored).

  It`s not feasible to give any preferred type for this structure at this phase of design, most probably it will be a combination of the above mentioned types.
- During the design, the final choice for the type of construction should be determined by an assessment of the following aspects:
    - extent to which functional and technical requirements are met;
    - risks;
    - ease of execution;
    - execution risks;
    - maintenance effort;
    - recovery options;
    - influence on the environment; and
    - total costs.
- A rough estimation of the area (length) of scour protection for this spillway could be about 50-100m at the upstream side of the spillway and 150-300m downstream of the spillway.
- To present another alternative, it is also an option to reduce the amount of energy of the water by creating a stilling basin at the downstream site of the spillway. An indicative example is shown in the figure below.

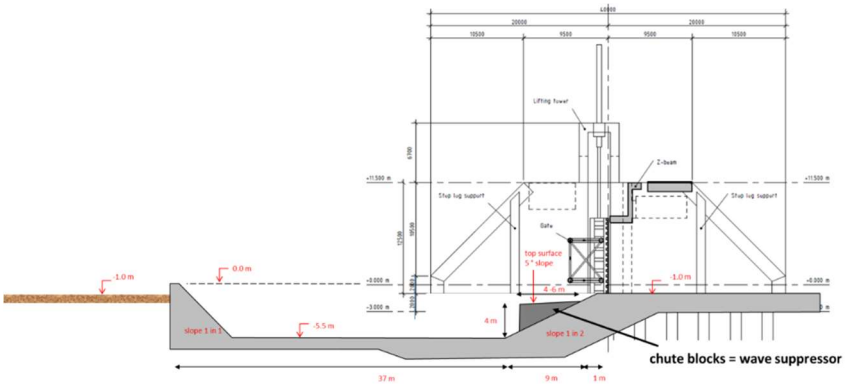


Figure 2-4: Indicative spillway stilling basin

2.7 Risks

Based on the above review of the permanent Kalpasar Dam structures, we perceive the main risks related to the design of the permanent structure of the Kalpasar Dam to be:

- Design conditions such as design wave height and design water level. These parameters can have considerable influence on the armour layers and the crest level elevations, and were uncertain at the time of review. It is noted however that design water levels and armour layer considerations were agreed in a Kalpasar project meeting of 12 November 2022, which has now mitigated this risk, enabling the design to be further developed on basis of these agreed parameters.
- Soil conditions. The soil conditions can lead to geotechnical instability and settlement of the dam structure. This can have a large impact on the design and the cost of construction.
- The feasibility of constructing the armour layers of the dam. A detail study is considered essential to determine the construction method possibilities and how the large quantities of armour layers needed for the dam can feasibly be produced in the envisaged timeframes.

2.8 Considerations for further development and optimisation

Based on the above review, we consider that the project would benefit from the further development and optimisation of the following aspects:

- Metocean study to better determine the design wave height and design water level
- Metocean study of the reservoir with the dam constructed to determine design conditions at the reservoir side of the dam.
- Offshore to nearshore transformation of the design conditions to the Kalpasar Dam, both at the seaside of the dam and at the reservoir side of the dam.
- Multi criteria analysis of concrete armour elements to be used, including optimization of the required sizes.
- Design of the required stone gradings, armour elements and crown wall elements considering the agreed design conditions.
- Detailed study of the required crest levels and reclamation levels.

- Consideration of all possible load combinations to confirm the mentioned governing load cases statements.

2.9 Conclusions

The reference design of the main dam of the Kalpasar Dam was checked according to the applicable formulae from the Rock Manual. The design was found to be sufficient for the boundary conditions provided by the client. This has been validated already by the model tests performed by CWPRS. In a next design phase, the boundary conditions should be more thoroughly researched.

Volumes of the materials needed for construction have been determined from the conceptual design of the Kalpasar Dam. From these volumes a cost estimate can be made for the construction of the main dam.

3 Spillway gates – Selection of type and operational mechanism

3.1 Alternatives for gate type

This section contains a study about the most suitable type of gate which will be implemented for the spillway structure.

3.1.1 Boundary conditions and design requirements

- The water levels are described in Section 2.2.
- The discharge is described in Section 3.1.2
- The option for passing navigation is excluded for this design.
- The total movable length of the spill way is assumed to be 50m wide

The main functional requirements of the gates are:

- Flood defence
- River defence
- Discharge of water
- Regulating of water
- Separate salt/fresh water.

3.1.2 Discharge capacity

The discharge capacity is a vital design condition for the discharge sluices. The total discharge capacity follows from three parameters: Under which conditions the water can be discharge into the ocean. Secondly, the amount of water which can be stored in the reservoir during such an event. In other words, by how much the water level is allowed to increase during a storm event. Finally, the design flood of the river system. This is amount of river- and rainwater which will reach the reservoir during a storm.

The department of Hydrology of the Indian institute of Technology Roorkee (ITT Roorkee) performed a study to determine the discharge capacity required for the discharge sluices [3]. This section provides a summary of their findings. The summary intents to provide some additional background on the discharge capacity and should not be interpreted as a review of their findings.

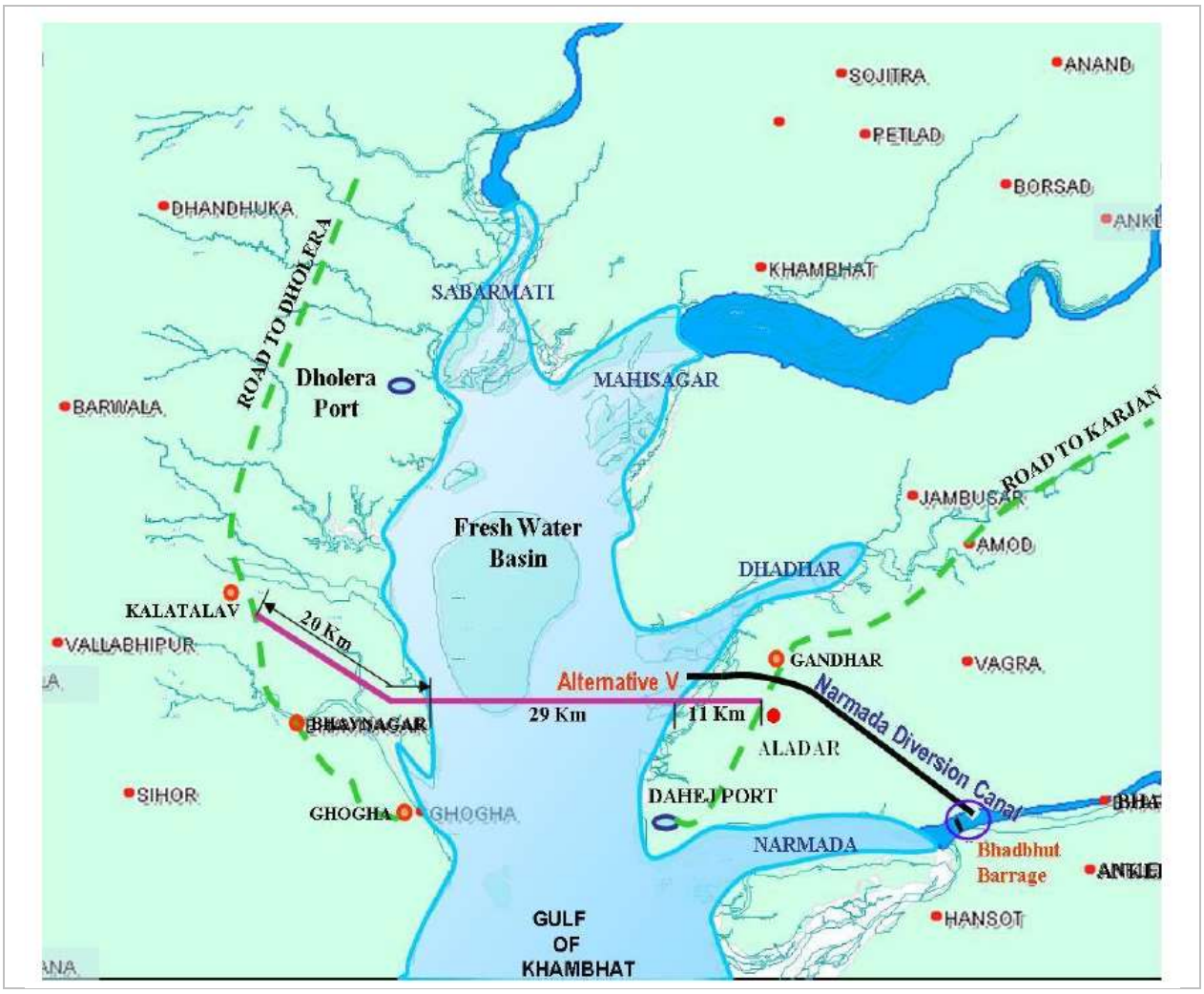


Figure 3-1: General layout of the Kalpasar project. Figure copied from IIT Roorkee, (2018).

One of the three parameters which determine the discharge capacity are the conditions under which the water can be discharge into the ocean. IIT Roorkee (2018) assumes that the water can only be discharged from the reservoir when the reservoir level is higher than the tidal level plus an additional 0.2 m to make sure that the water level differences can be verified by the naked eye during operation of the gates. The crest of the spillway is at -3.5 m MSL, which implies a maximum head over the spillway of 8.5 m as the maximum water level in the reservoir is restricted to 5 m MSL.

The second parameter is the storage of fresh water in the basin itself during a storm event. ITT Roorkee (2018) states that the maximum water level in the reservoir is restricted to 5 m MSL. Additionally, they take the highest operational level at 3 m MSL. This level is used at the start of a storm. During a storm the water level is allowed to rise by 2 meter.

The study of ITT Roorkee (2018) focusses on the design flood for the Kalpasar project. The design flood was derived using a deterministic approach based on the unit hydrograph theory for different rivers flowing into the Kalpasar reservoir, see Figure 3-2. The Kalpasar reservoir receives water from the Narmada diversion channel, the rivers Dhadhar, Sabarmati, Mahi and some small rivers in the Saurashtra region on the west coast (Wadwan Bhogavo, Limbdi Bhogavo, Sukhbhadar, Utavali, Keri, Ghelo and Kalubhar). The

discharge into the reservoir is a combination of free flow from the catchment areas between control structures and the reservoir and water released by control structures upstream.

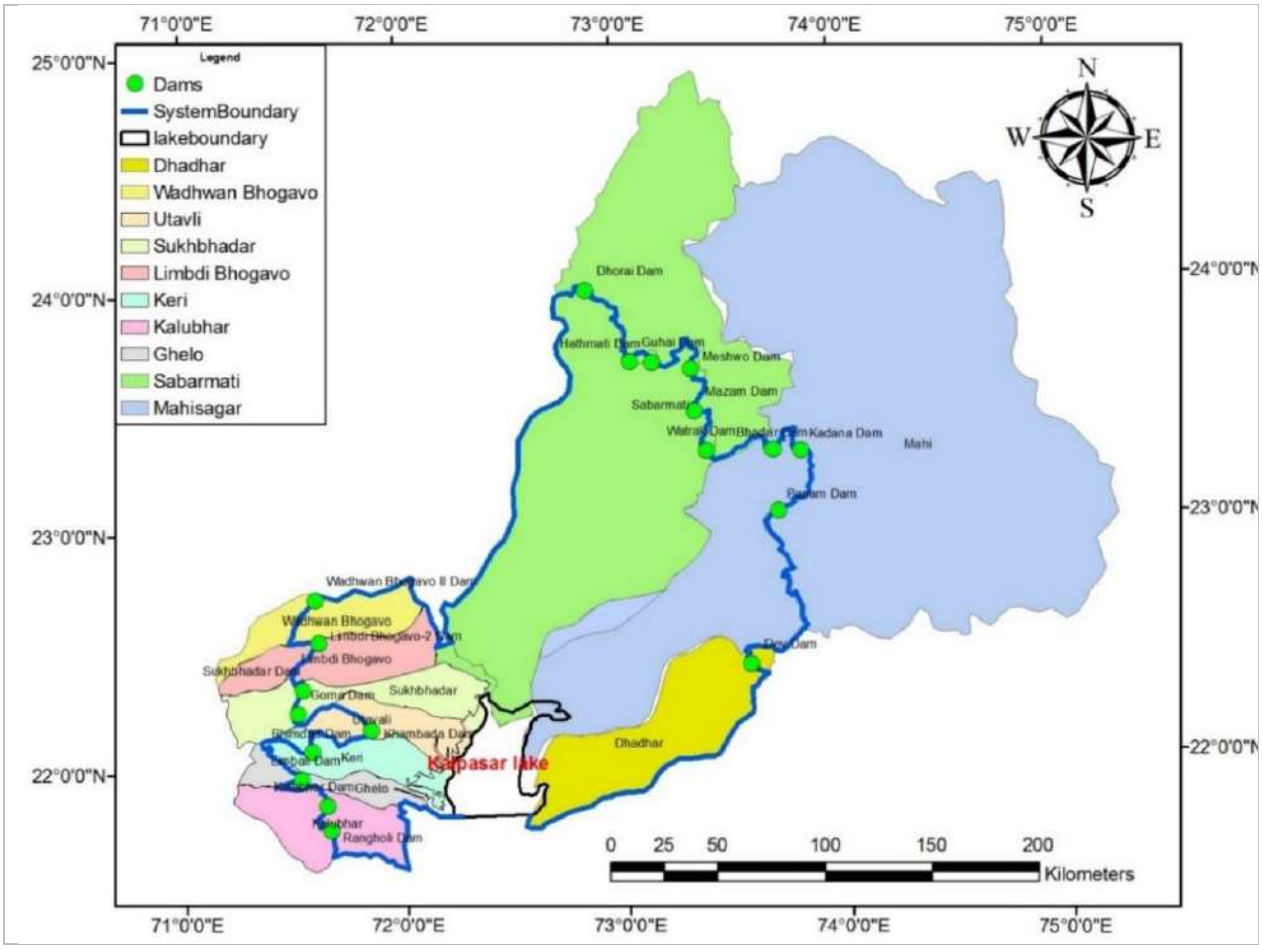


Figure 3-2: The system boundary of the design flood including various dams upstream. Figure copied from IIT Roorkee, (2018).

The flood routing into the reservoir was done using the Muskingum method. For each river basin, i.e. Dhadhar, Mahi, Sabarmati, Saurashtra region, the total flood volume into the reservoir was determined from the Probable Maximum Flood (PMF) hydrographs and the water discharged by the dams in the respective basins. These results were then used to assess the critical flood volumes for two types of storms: (1) a storm centred over Dhadhar, Mahi and Sabarmati basins and (2) a storm centred over the Saurashtra region. In both cases, the total flood volume was a combination of the total PMF volumes of the basins over which the storm is centred, 50% of the PMF volumes of the other basins, and the volume of water which felt directly over the Kalpasar reservoir. The results showed that a storm centred over Dhadhar, Mahi and Sabarmati basins leads to the highest flood volumes. Therefore, the flood characteristics of this kind of storm were adopted for the design of the spillway capacity.

It is important to realise that the design storms had a maximum duration of 3 days. This duration was chosen under the assumption that the various dams upstream are properly managed. A cascade failure event, a dam higher upstream fails during a flood event, was not considered.

The required discharge is determined by simulation a 200 hours timeseries of the design flood. As boundary condition on the sea side, the tidal level after construction of the dam was used. The most

critical 200 hours of the tidal data were selected. This selection was such that the water level in the Gulf of Khambhat is higher the water level in the reservoir (3 m MSL) and rising at the start of the PMF. This to obtain a critical situation in which water cannot immediately be released from the reservoir but first has to be stored for some time. Furthermore, it was assumed that the gates are closed after 168 hours and all incoming flood water has to be stored in the reservoir for the last 32 hours of the design storm. The simulations revealed that the required maximum discharge of 110.000 m^3/s is required to keep the water level inside the Kalpasar basin under the 5 m MSL.

3.2 Design options for the spillway gates

This paragraph describes several options for the gate design. Various gate options listed are taken from *"Lock Gates and Other Closures in Hydraulic Projects-2018".*

3.2.1.1 Vertical lifting gates

A vertical lift gate represents one of the oldest and simplest ways to hold a differential water head. Its historical roots reside in irrigation systems, but it is also widely used in river weirs, navigation locks and flood barriers nowadays. The simplicity of the very idea of this gate type, certainly in terms of risk management, is its biggest advantage. It results from its relatively short and easy to realize linear movement that is required to open and close the gate.

The common properties of all the systems of vertical lift gates are:
- plane structure located above water when opened;
- linear, vertical movement between the open and closed mode; and
- hydraulic load transfer by flexural bending.



Figure 3-3: Vertical lifting gates in the Hartel Barrier, The Netherlands



Figure 3-4: Nakdong Barrier (South Korea), a combination of vertical lifting gates, double leaf gates and (small) navigation lock.



Figure 3-5: Oosterschelde Barrier (The Netherlands): Vertical lifting gates



Figure 3-6: Youngsan Barrier (South Korea): vertical lifting gates

3.2.1.2 Sector gates (horizontal axes)

Sector gates rotate around a horizontal axis. The gate can retain water upstream or downstream. Ships can pass the gate in open position. Commonly, rotary segment gate are used for flood barriers, known from the publicity and prominent location of the London Thames Barrier. The operation as a lock gate generates, however, other requirements than that as a flood barrier. The difference lies, for example, in the frequency of closing and opening. Flood barriers are not closed frequenctly.



Figure 3-7: Sector gate with horizontal rotation axis



Figure 3-8: Thames Barrier (United Kingdom) Set of sector gates. (Navigation possible)



Figure 3-9: Ems Barrier, Germany: Combination of vertical lifting gate and a sector gate (for navigation).

3.2.1.3 Sector gates (vertical axis)

Vertically hinged sector gates are used on storm surge or hurricane protection sites, and on navigation locks. Their primary advantage is that they can operate with differential head on either side of the gate. The sector gate consists of two gate leaves, each made of a curved skin plate, with a framed structure linking the skin plate back to a point of rotation located at the skin plate's center of curvature. When open, the gate leaves are within recesses at either side of the navigation channel. The large recess necessary is one primary disadvantage of a sector gate structure. The concentrated loads from the gates result in heavy foundation structures.



Figure 3-10: Sector gate with a vertical axis (Maeslant Barrier, The Netherlands)

3.2.1.4 Visor gates

A visor gate is not a common gate. However, this type of gate typically requires economical use of construction material. A visor gate is constructed in such a way that the retaining water results in axial forces (tension or compression) in the structure and not bending moments. Typically, structures transferring axial forces are lighter compared to structures transferring flexural bending moments. As a result, predominately tensed or compressed structures require substantially less material compared to structures designed to resist hydraulic loads through bending.



Figure 3-11: Visor gate at Driel, The Netherlands

3.2.1.5 Radial gate

A radial gate is a gate that rotates about a horizontal axis and consists of a circular skin plate supported at each side to trunnions by two or more radial struts. The trunions are in the gate rotation axis. This design feature works to focus the resultant hydrostatic loads acting on the surfaces of the gate skin plate through the trunnions. The advantage of this arrangement is that there is no leverage between the resultant hydrostatic load (acting on the skin plate) and fulcrum of the gate. This prevents moments caused by the hydrostatic loads, which otherwise would have to be resisted by the structure or its hoist machinery.



Figure 3-12: Radial gate Vortex Hydra Dam

3.2.1.6 Inflatable gates

Inflatable gates provide an alternative to traditional steel or wooden gates. They are used throughout the world, extensively in Japan and by the worldwide hydropower industry. One primary use of inflatable gates is for small to medium size, run-of-the-river, hydroelectric plants. The inflatable gates are often rubber gates and are installed to replace older flashboard systems, or to increase the depth of an impoundment and provide crest control. At these facilities, the gate is fully inflated most of the time while pool elevation is controlled by turbine settings. Inflatable gates at hydropower structures have generally worked well at passing ice and debris.



(a) (b)

Figure 3-13: Rubber gates with flow breakers: (a) toothed seam at Highgate Falls gate, Vermont, USA and (b) flow breaking corbels at Marklendorf RG, Germany.

3.2.1.7 Flap gates

Crest gates belong—together with vertical lift gates and single-leaf (or swing) gates—to the oldest types of hydraulic closures used by men in free surface waters. Flap gates are mounted with a hinged connection at the crest or weir sill of a spillway. Today, most hinged flap gates are raised to achieve a closed position and lowered to achieve an open position by a more sophisticated drive system than their buoyancy. As they are lowered, they conform to the weir sill of the dam. They can be designed to allow overtopping flow at any height through their range of travel. This functionality is most commonly used to provide a method of maintaining required navigation depths during periods of low river flow.



Figure 3-14: Flap gate

3.2.2 Preliminary elimination of gate types

Regarding the design of the gates a multi criteria assessment has been carried out in order to find the best option for the gate type and global dimensions for the spillway structure in Kalpasar. This assessment includes a first elimination of gate types and a further comparison of the remaining gate types.

Sector gates (vertical axis). General, these types of gates will function well for flood defence, but these types are not very suitable for discharging water. A very concentrated flow of water with very high speed will occur while opening or closing the gates. This water behaviour will have an enormous effect on the concrete sill structure and scour protection.

Sector gates (horizontal axis). These gates are in general sensitive for sedimentation. The needed recess in the floor of the gate and the space behind the moving wheels are the locations where sediment will be disposed. The sealing of these types of gates is rather complicated and the performance of complete water tightness is difficult to create. The advantage of a sector gate is that by opening there is an unlimited height for passing ships. However, for this spillway design, navigation is not a primary issue.

Inflatable gates: These types of gates are suitable as flood defence, but not suitable if they have to operate (inflate/deflate) frequently in short periods.

Visor gates. In general visor gates are very suitable as river water levelling structures but have never been built as a flood defence.

The most suitable types of gates are the remaining radial gate, flap gate and lifting gate.

3.2.3 Comparison of suitable gate types

A comparison of the three most suitable types of gates has been elaborated in this paragraph.

- Radial Gate
- Flap gate

- Lifting gate

The criteria for comparison and the weighting factors are listed in
Table 3-1:. In this table the weighting factors are focussed on functional and maintenance aspects of each type (50%).

In the following phrases the differences in appreciation for the criteria as mentioned in the table are further briefly explained and additional information is added.

3.2.3.1 **Functionality**

Flood defence
There is no significant difference between the gate types regarding the functional requirement of flood defence.

River defence
Regarding this function there is also no significant difference, and all the three gates perform well.

Total discharge
Regarding this function there is also no significant difference.

Regulating discharge and level control lake side
The flap gate is very suitable for regulating a constant water level at the lake side. Therefore, a higher score is awarded to this type. However, this function can be integrated in both the radial and lifting gate.

Separate salt/fresh water
Both the radial gate and the flap gate have a rather complicated and sensitive sealing at both vertical sides of the hinges. This vertical sealing is sensitive to sedimentation and measurement imperfections will easily lead to leakage.

Navigation
All the three gates have limited height for the passing vessels. In this project however this is not considered relevant since the crossing bridge for regional traffic creates a limitation on the passing height.

3.2.3.2 **Availability**

Navigation
Potential ship collision is not considered in this project.

Sedimentation
The radial and flap gate are more sensitive for accumulation of sediment which can cause possible malfunction of the barrier.

Debris
The lifting gate is vertical steel plate with a three-dimensional truss work (framework). Floating debris can more easily accumulate in this frame than the radial gate.
Non floating debris will have the most severe effect on the flap gate and therefore this type gets the lowest score.

3.2.3.3 Reliability

The reliability of the gates depends mostly on the functioning of the operating mechanism. Assuming that the gates are equipped with the same type of operating mechanism, there is no significant difference.

3.2.3.4 Technical aspects

Concrete sill structure.
The lifting gate has the simplest concrete sill, a horizontal floor. The flap gate needs a more complicated concrete structure.

Sealing
The lifting gate has the simplest way of horizontal and vertical sealing and becomes therefore the highest score. The sealing of the radial gate and the flap gate are more complex.

Operating mechanism
The radial gate has rather big horizontal pivots with bearings which have to deal with high concentrated loads. These constructions need very precise installation and secure maintenance. The flap gate has supports hinges in under water position which are difficult to inspect and maintain

3.2.3.5 Environmental aspects

Risk of pollution and noise.
From the gates (steel structures) there is no significant risk of pollution. The steel gates can be recycled at the end of their life. Pollution is more critical for the operation mechanism.

Noise
Regarding this aspect all the three gate types of are equally valued.

3.2.3.6 Costs

Investment costs
It is expected that the lifting gate is more expensive due to the fact that this type needs lifting towers.

Maintenance costs
Due to the complex shape and sealing we expect that the radial and flap gate need the most maintenance and the lifting gate will need less maintenance because it is the simplest structure with a simple sealing.

Life cycle costs.
The maintenance aspects influence the total life cycle costs and the difference in score.

3.2.3.7 Maintenance

Frequency of major overhaul
Because of the above-mentioned aspects regarding sealing, we expect that the lifting gate has the lowest frequency and therefore the highest score.

Qualification of maintenance personnel
In this case there is no significant difference in the required qualification level of personnel.

Experience in India

As far as known there is not a significant difference in experience between the three types of gates.

Aesthetic aspects

The validation of these aspects is subjective, but we think that there is no significant difference between the three types. General, barriers and weirs will not score on esthetical aspects. The structure itself is partly or complete under water during operation and hardly visible.

Table 3-1: Comparison suitable gate types

			Type of Gate:					
#			Radial Gate		Flap gate		Lifting gate	
	Criteria	Weight factor W (%)	Appreciation A	W x A	Appreciation A	W x A	Appreciation	W x A
1	**Functionality**	**40**						
	Flood defence	8	8	0.64	6	0.48	8	0.64
	River defence	8	8	0.64	9	0.72	8	0.64
	Total Discharge	8	8	0.64	7	0.56	8	0.64
	Regulating Discharge	8	8	0.64	7	0.56	8	0.64
	Separation salt/fresh water	8	6	0.48	6	0.48	8	0.64
	Navigation	0	7	0	7	0	7	0
2	**Availability**	**10**						
	Navigation	0	7	0	6	0	7	0
	Sedimentation	5	6	0.3	6	0.3	9	0.45
	Debris	5	7	0.35	7	0.35	6	0.3
3	**Reliability**	**5**						
	Driving equipment	5	7	0.35	7	0.35	8	0.4
4	**Technical aspects**	**15**						
	Concrete sill structure	5	6	0.3	5	0.25	9	0.45
	Sealing	5	6	0.3	6	0.3	9	0.45
	Operating mechanism	5	7	0.35	7	0.35	8	0.4
5	**Environmental aspects**	**5**				0		
	Risk of pollution of the water	3	8	0.24	8	0.24	8	0.24
	Noise	2	8	0.16	8	0.16	8	0.16
6	**Costs**	**10**						
	Investment costs	4	7	0.28	7	0.28	6	0.24
	Costs of maintenance	3	7	0.21	6	0.18	8	0.24
	Life cycle costs	3	7	0.21	6	0.18	9	0.27
7	**Maintenance**	**10**						
	Frequency of major overhaul	5	6	0.3	6	0.3	8	0.4
	Qualification required of maintenance personnel	2	7	0.14	7	0.14	7	0.14
	Experience in India	3	7	0.21	7	0.21	7	0.21
8	**Aesthetic aspects**	**5**						
	Visual aspects	2	7	0.14	8	0.16	8	0.16
	Suitable for landmark	3	7	0.21	8	0.24	8	0.24
		100						
	Total appreciation (weight * appreciation)			**7.1**		**6.8**		**8.0**

3.2.4 Recommended gate type

The result of this assessment is that for this project a vertical lifting gate is the most suitable solution. The result is based on the weighting factors which focus on functionality and maintenance. The appreciation of the three types of gates is mainly based on expert judgement and experience. The main advantage of the lifting type of gate is the relatively simple steel structure with a simple sealing system. Reducing of maintenance costs will be a benefit consequence.

3.3 Gate operational mechanism

3.3.1 General

The sections below give a short description of the mechanical installations for the gates of the spillway. The spillway will be carried out with lifting gates. For the lift operations several options are possible. The most suitable systems to use for these large vertical lifting gates are:

- electromechanically with winches
- hydraulically with large cylinders
- hybrid with a combination hydraulic/mechanical.

3.3.2 Boundary conditions

Main dimensions and mass for the spillway gates:

•	width of the gate	:	50	m
•	height of the gate (+8.0 m to -12.0 m)	:	20	m
•	water difference (+4 m to -4.0m)	:	8	m
•	Approx. mass of the gate	:	700	ton
•	lifting height (20 m opening + 2 m free height	:	22	m

3.3.3 Gate details

3.3.3.1 Counterweights

The required power when using a lifting gate without counterweights is high. The spillway is not located in an industrial area and therefore the available power connection might be limited.

When lifting a gate, the lifting force consists of the weight of the gate and the resistance the gate will be exposed to due to friction and hydraulic forces. Because the gate will be lifted under limited water level differences, the hydraulic forces on the gate are low and the friction forces likewise. For the power estimate for the basic design a resistance force equal to 10% of the gate weight is assumed. In the detail design phase the forces will be determined accurately.

3.3.3.2 Lifting velocity

The lifting height of the gate is about 22 m (see assumption above). For the spillway gates we propose a lifting velocity of 100 mm/s. The number of lifting operations is low, only a couple of times per year and the needed power is proportional with the velocity, for which a high velocity is not required. With the chosen lift velocity, the motion time of the gates will be about 4 minutes.

When assuming an efficiency of 70% for the operating mechanism, the needed power to be found when not using counterweights will be about 1200 kW. The found power demand is far too high to defend the choice of unbalanced lifting gate as an appropriate choice. Therefore, the use of counterweights is recommended.

3.3.3.3 Elected overweight

A precondition is that the gates can always descend on their own weight. This implies a needed minimum overweight. Now friction cannot be neglected to have the ability to close the gate under gravity (normal of emergency depending on the choice of the type of operating mechanism).

The lifting gate will be designed as an open structure without air boxes/cabinets. Because of the buoyancy the lifting force with the gate submerged will be less. Under the assumption, when opening the gate under extreme conditions, the gate will be submersed for 90% (conservative consideration); the lifting force will be 80% less than when lifting above water (0.9·(7850-1000)/7850)). With 7850 kg/m3 for the density of steel and 1000 kg/m3 for the density of water.

For the resistance the assumption of a force equal to 10% of the weight of the gate, the imbalance is not allowed being less than 70 - 100 ton. For the basic design we therefore propose counterweights with a mass of about 600 ton (2 times 300 ton).

3.3.4 Operating mechanism

There are several options to operate the gates. The most suitable systems to use for these sized gates are:

- hydraulically with large cylinders
- electromechanically with winches
- hybrid system with a combination hydraulic/mechanical

3.3.4.1 Hydraulic system

For the dimensions of the hydraulic cylinders, we use a lifting force of about 2000 kN, build up from:

- lifting force due to overweight : 700 kN
- resistance (10% of gate weight) : 700 kN
- load factor (permanent load) : 1.35 m

With a force of 1000 kN per cylinder and the following assumptions:

- maximum allowable pressure cylinder : 120 bar
- surface ratio (A_{piston} / A_{rod}) : 2

the needed piston diameter will be about 500 mm, assuming cylinders mounted in pulling position. I.e. with the piston rod directed downwards.

With for the cylinder a ‘zero-length’ of about 5 times the piston diameter and a rest length of 250 mm at both sides (rod side and bottom side), the length of the cylinder jacket will be about 25.0 m. In extended position, the cylinder has a length of about 50 m. When using cylinders with trunnion mounting close to the cylinder head, the cylinders can be placed on the towers with a height close to the top of the gate in upper position. The jacket/casing of the cylinder will be protruding about 22 m above the tower.

3.3.4.2 Electromechanical

Normally with lifting gates, winches will be used. There are several options for the operating mechanism. For example, the motion cable can be attached to the gate itself (left figure below), or can be attached to the counterweights (right figure below). When connected to the gate, the winches will be normally placed in the top of the tower; when connected to the counterweight, the winches will be placed in the basement of the hoisting towers.

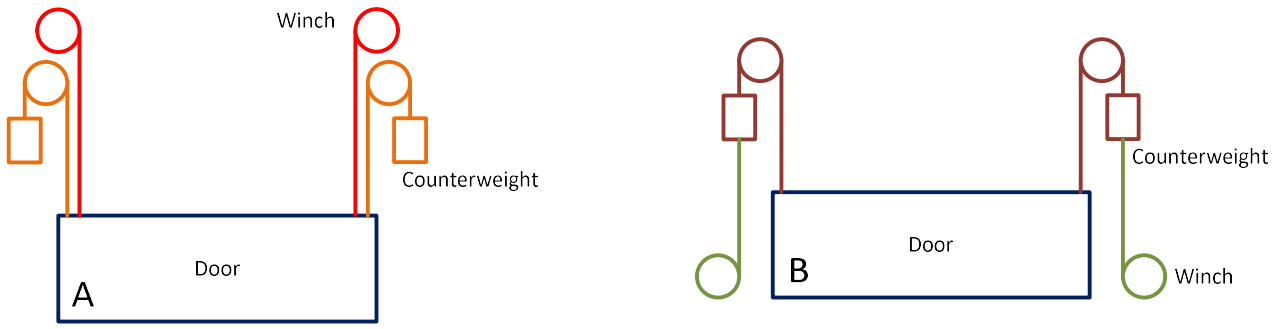


Figure 3-15: principle of hoisting mechanism for the gate

In both cases the load the vertical load of the towers, although different composed, will be 14700 kN; 2.1 times the weight of the gate (resistance assumed 10%). Per tower 7350 kN (without load factors)

3.3.5 Considered lift systems

- var A : Hydraulic system with two cylinders; the cylinders with the piston rod upwards. Two hydraulic units, each in one tower. The hydraulic systems provided with a controller for synchronous motion. Needed height of the towers ca 40 m high.
- var B : Hydraulic system with two cylinders; the cylinders with the piston rod downwards. Two hydraulic units, each in one tower. The hydraulic systems provided with a controller for synchronous motion. Needed height of the towers ca 25 m high when using cylinders with trunnion mounting close to the cylinder head.
- var C : Electromechanical system with two winches. A winch on each side of the gate, placed in the tower or a basement below the tower.
  The winch systems provided with a controller for synchronous motion.
- var D : Electromechanical system with one winch and a cable running through the gate to the other side of the gate. The winch placed in the tower or in a basement below the tower. In order to keep the gate in straight position straightening cables are needed (not shown).
- var E : Electromechanical system with a toothed rack. The toothed rack places in the towers, the gate will pull itself up with a pinion gear on both sides of the gate. The machine work to drive the pinion gears built-in the gate. The pinions star connected for synchronous motion of the gate or when driven by separate drives using a controller for synchronous motion.
- var F : Hybrid system: cable system driven by one hydraulic cylinder; the cable runs through the gate to the other side of the spillway opening. In order to keep the gate in straight position straightening cables are needed (not shown). By using multiple cable parts, the cylinder stroke can be reduced (1/2x), consequently the cylinder force will increase (2x).
- var G : Hybrid system: two cable systems each driven by a hydraulic cylinder. The hydraulic units with cylinder and winch in each tower. The hydraulic systems provided with a controller for synchronous motion. By using multiple cable parts, the cylinder stroke can be reduced (1/2x), consequently the cylinder force will increase (2x).

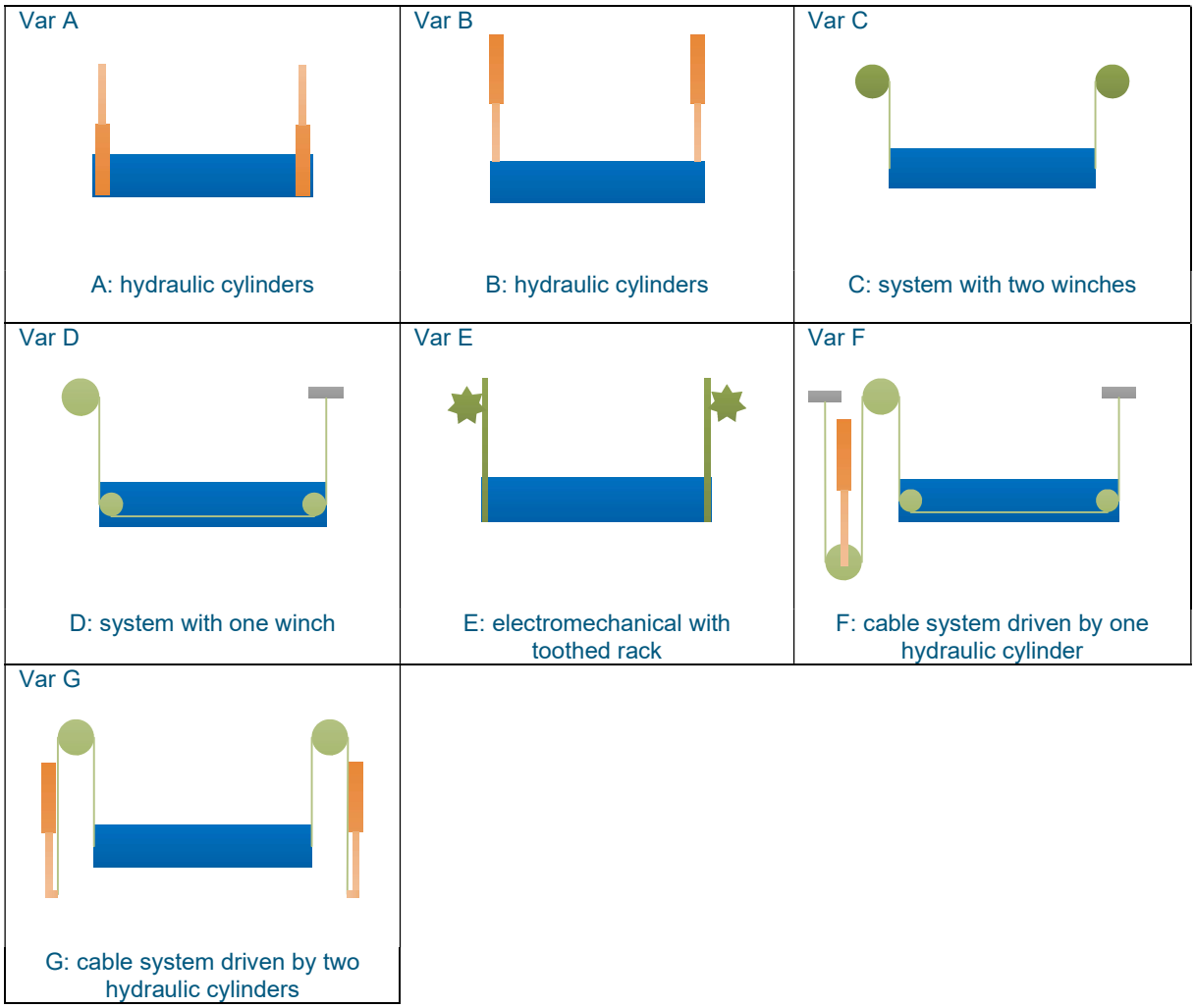


Figure 3-16: considered lift variants

Not considered:

- var H : Electromechanical system with star connected winches, build in a 'bridge' on the towers above the gate. Because of the star connection the winches will rotate synchronously. The system will not be considered because of civil limitations.
- var I : Electromechanical system with star connected winches, build in a 'tunnel' under the foundation floor. Because of the star connection the winches will rotate synchronously. The system will not be considered because of civil limitations.
- Var J : Hybrid systems with winches driven by a hydraulic motor. The system will not be considered because the principle is equal to winches electrical driven and therefore not determinative.

3.3.5.1 Provisional choice of the operation mechanism

For the purposes of the feasibility is examined what type of operation mechanism for the lifting gates the most appropriate mechanism is. In the further design phase, a more detailed consideration can be made. To get a well-considered choice of mechanism, a shortened and simplified Trade-Off Matrix is made with a limited number of criteria.

The alternatives are evaluated on the following criteria:

- investment

- operating and maintenance costs
- easy suitability into construction
- reliability and availability
- simplicity
- suitability
- damage on a collision.

Weighting factors

In order to weigh up the importance of the different criteria, a weighting factor in the assessment is used for the options. For the determination of the weighting factors, all criteria are weighed out with each other. The points-score found gives hereby the ranking of the criteria.
In order to influence the score not excessive by the weighting factors, the most important criteria are weighted with factor of '3' and the least important with a factor '1'. The table below shows the assessment and the choice of the weighting factors.

Table 3-2: Weighing factors

No	Criteria																				sum	WF
1	investment	1	1	0	1	0	0														3	2
2	operating and maintenance costs	0						1	0	1	0	1									3	2
3	easy suitability into construction		0					0					0	0	0	0					0	1
4	reliability and availability			1					1				1				1	1	1		6	3
5	simplicity				0					0				1			0			0	0	1	1
6	suitability					1					1				1			0		1	5	3
7	damage on a collision						1					0				1			0		3	2

Assessment of the variants

With the assessment following ratings are used:
- 2 = good
- 1 = adequate
- 0 = moderate.

The table below shows the assessment of the variants and the weighted valuation. The best variant to be found is the operation mechanism with two winches (variant C) with a score of 25 out of 28. A condition for a suitable solution for the variant to use variant, is a score of at least 70% of the ideal score. With a score of 89% this is the case. From the other variants, only variant D meets up the minimum score with 71%. For the basic design variant C will be proposed, although for the winch system a complex controller for synchronous motion is needed.

Table 3-3: Assessment of variants

no.	criteria	WF	ideal	valuation							weighted valuation						
				A	B	C	D	E	F	G	A	B	C	D	E	F	G
1	investment	2	4	1	1	2	2	1	0	0	2	2	4	4	2	0	0
2	operating and maintenance costs	2	4	2	2	1	1	1	1	0	4	4	2	2	2	2	0
3	easy suitability into construction	1	2	0	2	2	1	1	0	0	0	2	2	1	1	0	0
4	reliability and availability	3	6	1	1	2	2	1	0	0	3	3	6	6	3	0	0
5	simplicity	1	2	1	1	1	2	1	0	0	1	1	1	2	1	0	0
6	suitability	3	6	1	2	2	1	1	1	1	3	6	6	3	3	3	3
7	damage on a collision	2	4	0	1	2	1	0	1	2	0	2	4	2	0	2	4
	total		**28**	**6**	**10**	**12**	**10**	**6**	**3**	**3**	**13**	**20**	**25**	**20**	**12**	**7**	**7**

A : hydraulic system with two cylinders; the cylinders with the piston rod upwards
B : hydraulic system with two cylinders; the cylinders with the pistonrod downwards
C : electromechanical system with two winches
D : electromechanical system with one winch and a cable running through the door to other side of the lock
E : electromechanical system with a toothed rack
F : hybrid system: cable system driven by one hydraulic cylinder; the cable runs through the door
G : hybrid system: two cable systems each driven by a hydraulic cylinder

WF: weighting factor

valuation
2 = good
1 = adequate
0 = moderately

Short motivation

The section below gives a short motivation of the assessment.

Investment:
Variant C and D score the best on the investments. The used technics are standardized and commonly used in other applications. The toothed rack (var E) and the operation mechanism with the hydraulic cylinders have a lesser score because they are not standard and must be a custom-made product. The hybrid variants have the lowest scores because they are also custom made but also have the additional cable work in the system.

Operation and maintenance costs:
Variant A and B score the best on operation and maintenance cost. The hydraulic unit is very easy to maintain in a in an area with adequate space for maintenance. The cylinders outside are of low maintenance. The variants C, D and F score lesser because of the maintenance needed for the cable system. A lot of maintenance is needed on places difficult to reach to grease the cables. For variant E counts the same. The toothed rack also needs a lot of maintenance because of the greasing intervals. Variant G has the lowest score. The hybrid mechanism has a lot of components which need together with the cable systems a lot of maintenance.

Easy suitability into construction:
Variant A, F and G have the lowest scores. Variant A because of the extra high tower needed for the cylinder. Variant F and G because an extra chamber must be made for the cylinder(s). Variant C scores the best because the machine work with cable system can be relatively easy be incorporated in the civil structure. Variant D and E score adequate. Variant D because cables will have to run through the gate (motion and straightening cables). Variant B because of the machine work placed in the gate.

Reliability and availability:
The electromechanical operation mechanism with winches (variant C and D) will be considered the most reliable and because of the standard equipment also to have the highest availability. The hybrid systems have the lowest score on reliability and availability. The mechanisms have a lot of components which can fail, are custom made with long delivery times. All the other variants score adequate.

Simplicity:
Variant D has the highest score. The mechanism is very simple with only on winch system. A straightening cable will be needed for synchronous motion of the gate, but no separate control system is needed. Variant F and G have the lowest score because of the double system for the motion (hydraulics and cables). All the other variants score adequate. Often a control system for synchronous motion of the gate is needed.

Suitability:
Variant B and C have the highest score. Variant C, because the simplicity and because they are already often used as operation mechanism for lifting gates. The difficulty will be the controller for synchronous motion of the gate. Variant B is also often used for locks and movable weirs. The main disadvantage is the needed length of the cylinders. All the rest scores adequate.

Damage on collision:
Variant C and G have the highest score. Because the gate on two cables hangs, the operation mechanism is quite good protected by a collision. Variant A and E have the lowest score. Variant A because the cylinders which are partly incorporated into the steel structure of the gate. In a collision, it will be well

possible that the cylinders will be damaged. The same counts for variant E whereby the machine work will be built in into the gate. All the other variants have a score adequate.

3.3.6 Electromechanical system with two winches

Using the multi-criteria analysis provides that variant C, the electromechanical system with two winches, seems to be the best option.

For this phase of design, the principle as shown in the figure below will be proposed. Two counterweights on each side of the gate with a mass of about 300 ton each will be connected to the gate. With a mass of 700 ton for the gate, the overweight (above water) will be about 100 ton.

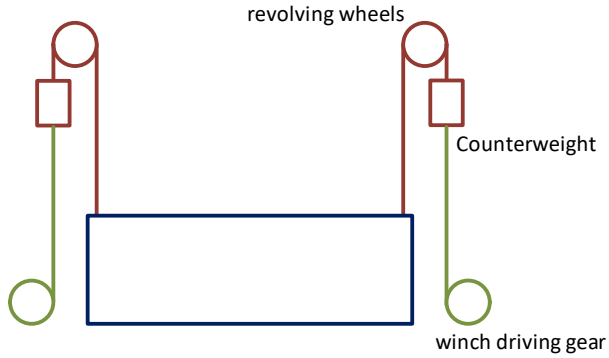


Figure 3-17: principle of moving gate

For moving the gate upwards and downwards, a winch system is connected to the counterweights. When pulling the counterweight downwards, the gate will be lifted.

4 Spillway gates - conceptual design of the lifting gate

4.1 Design of the lifting gate

The lifting gates can be designed lens shaped or with a flat retaining wall and arched shaped girders. To decide whether to prefer a lens shape or a flat shape type for the gate, the following aspects were examined:

-Design aspects
-Fabrication aspects
-Maintenance
-Functionality
-Esthetical aspects.
-Total costs

Design aspects.
The geometry of the gate:
The most economical shape for the gate is equal to a parabola, related to the bending moment. However, when the horizontal distance between the truss frame and the skin plate is identical, there will not be a big difference between the lens shape and the flat shape gate, regarding the needed amount of tons of steel. The lens shape gate is slightly in favour.

Vertical bearing.
The centre of gravity in a flat shape gate is more eccentric in respect to the vertical bearing than a lens shape gate. In order to lift the gate vertically the flat shape gate needs extra consoles to fix the hydraulic cylinders. The vertical bearing of a lens shape gate is much more in line and can be more easily integrated in combination with the horizontal sliding bearing. For a flat shape gate this aspect becomes more difficult to design.

Fabrication aspects.
Fabrication of plate and frame.
-The fabrication of a flat shape gate is simpler. The flat skin plate can be used as a horizontal reference surface where the truss frame can be mounted on.
However, to create an accurate plain straight surface for the skin plate related to a relative big distance is also not an easy task.
-For a lens shape skin plate a formwork must be built to create this shape.
-However small imperfections in measurements will not be visible because of the bow form.
The most difficult part to fabricate is the truss frame which is made of circular steel shape elements. This truss frame is identical for both gates.
Fabrication of bearing.
-As already mentioned in the design process the fabrication of the vertical bearing will be more difficult for the flat shape gate.

Maintenance aspects
-There are no significant differences between the two gates regarding maintenance. The same type of steel shapes are used in both the types. The amount of surface to be treated is also about the same.

Functionality aspects
-There is no significant difference between the two gate shapes. The most critical part is not the steel structure but in general the reliability of the driving mechanism, which determines the availability of the whole system.

Esthetical aspects
-When the gates are in function and must retain the maximum water level difference the maximum horizontal deflections will be approximately 30 mm. Although these deflections will hardly be visible it's more relevant that a flat gate should be absolutely flat, and imperfections will be visible. For a lens shape gate this aspect is hardly relevant.

Total costs
The total costs of the gates are not only influenced by fabrication, but also major determined by transportation and installation costs. Possible price differences in fabrication can be easily eliminated by high differences in transportation and installation costs.

Conclusion:
- A lens shape gate is considered as the most favourable option in this phase.

Optimum width of the lifting gate.
Foundation
There might be a possibility for a foundation without piles because of the preloading effect of the present soil. However, this possibility requires a rather rigid structure with mutual pier distances of about 40-50m equalizing the soil pressure as much as possible.
A greater span will concentrate the downward forces too much and will locally (at the piers and wing walls) increase the soil stresses causing unacceptable settlements. This might result in the necessity of a steel pile foundation and will increase the investment cost for the whole project considerably.

On the other hand, a larger span has advantages, including: easier and more cost-effective to prefabricate, less number of gates, easier and more reliable operations (better to open 1 gate than 3), more straightforward and reliable lifting mechanism, more cost-effective solution, easier to get watertight, less leaks around the doors.

Relation to bridge structure
The span of the gate is directly related to the span of the concrete bridge for the regional road. A span of about 40-50m would then be required for the bridge span, which is feasible in pre-stressed concrete. However, an important consideration is as to whether local contractors have experience with these large spans or would prefer to choose a standard span width that can be realized with prefabricated concrete girders. In this respect an optimization can be obtained in the span of barrier and bridge together.

A larger span for a bridge structure is not possible anymore with the use of prefab concrete girders. Introduction of a cable-stayed bridge, suspension or arch bridge type is necessary, which increases the costs enormous.

Summarized. A span of 50m is a feasible and cost-effective choice. Gates with a span up to 100m are already built, so there is no risk in the chosen dimensions of the gate, related to constructability.

Operating mechanism, Hydraulic cylinders
Frequently used in modern structures and applicable for heavy loads. An oil reservoir tank and a rather small capacity pump is needed for operation. The whole installation is simple, a redundancy system is easily installed which will create a higher percentage of availability.

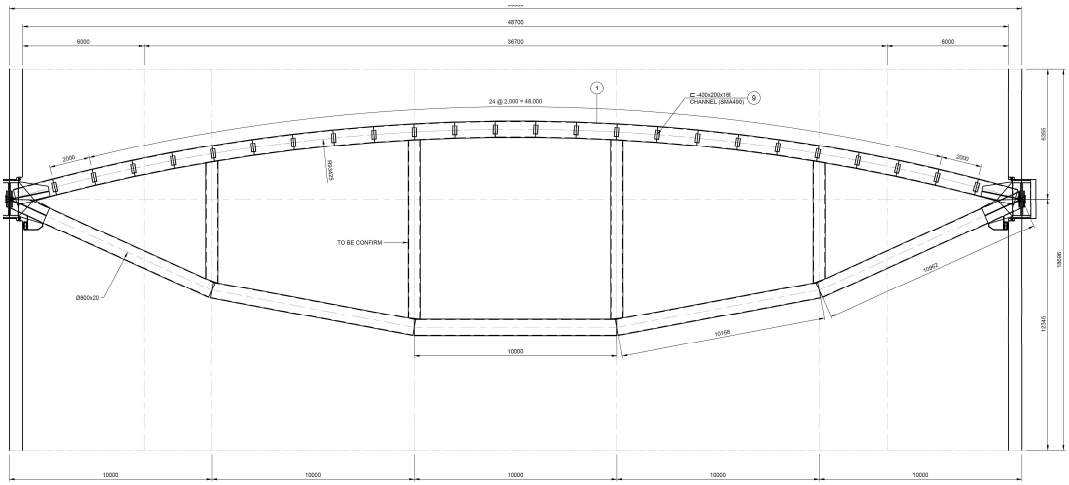
Watertightness
Watertightness of the gate will be achieved by using rubbers flaps at the walls and bottom to avoid leakage. Further to that, water levels differences will further reduce leakages as the doors will be pushed into their frame. In any case, leakage volumes will be very small, and sea water intrusion is unlikely to become an issue. These kind of details are not shown in this phase of concept, but are always feasible.

Corrosion protection
As a corrosion protection we recommend installing a robust coating system in combination with a cathodic protection system. Possibly some corrosion allowance can also be included if proven to be necessary in subsequent design stages. Skin plates are not so much relevant for corrosion protection, their main function is water retention. It`s also not so relevant which part of the gate is located in fresh or salt water, anyhow coating in combination with a cathodic protection system is needed.

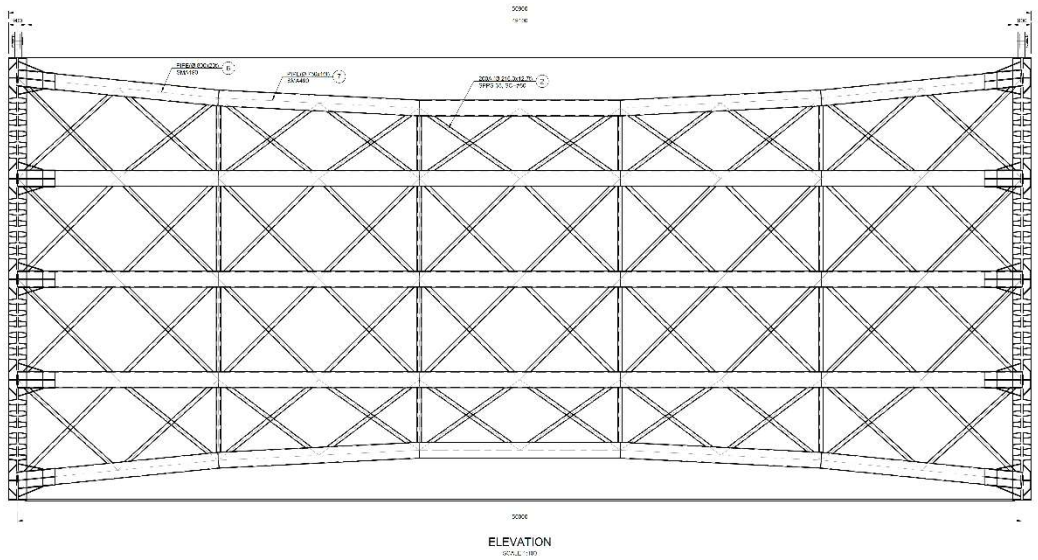
Gate installation
Gate installation and replacement will be executed by using marine equipment, for example by large crane vessels / pontoons lifting the gate from its position, placing it directly on pontoons, where maintenance is done. New gates (if applicable) can also be placed back using marine equipment. Further to this, the equipment (hoisting mechanism, cables, engine) should be accessible with doors and shafts allowing for maintenance and replacement – accessible for small cranes.
The risk of replacement of a complete gate during service lifetime is very small. Normal inspection and maintenance will occur when the gate is in position. It must be possible to dry out an opening by using stoplogs. The size and weight of these stoplogs components will be much smaller and can be placed in position with smaller floating equipment.

Support structure of the gate
The whole electromechanical system with two winches and contra weight, will be built in solid concrete towers. These towers will protect the operating system.
The support structure will consist of two wingwalls and a solid concrete floor slab. The operating mechanism will be stationed in two lifting towers of about 22m height as mentioned in the report volume 3. The horizontal dimensions of these towers will be about 4x4 m^2.

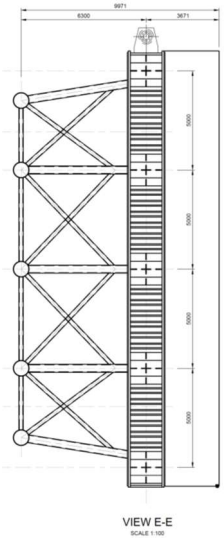
Design drawings
Design drawings for the lens-shaped lifting gates are provided in Appendix A. Excerpts from these drawings are provided below for illustration.



VIEW A-A (OPTION - 2)



ELEVATION



VIEW E-E

Figure 4-1: Excerpts from drawings of the lifting gate (resp; top view, elevation and typical vertical section)

4.2 Quantities of the lifting gate

A preliminary design of the lifting gates provides an indication of the steel sections used for the gate. Summarizing all steel sections provides the total weight. The cost of the gate can be estimated based on the weight of the gate.

Table 4-1: Structural sections lifting gate

#	Reference 1	/Structural section	Material
1	Tensile beams	pipes	S355/SMA490
2	Bracing in the tensile beams	pipes	S355/SMA490
3	Spacers	pipes	S355/SMA490
4	Bracing in the spacers	pipes	S355/SMA490
5	Horizontal beams - retaining wall	U shape girders 200*200	S355/SMA490
6	Vertical beams - retaining wall	T section 600*400	S355/SMA490
7	Compression beams - retaining wall	pipes	S355/SMA490
8	Cladding	flat plate	S355/SMA490
9	Support	various	S355/SMA490

Table 4-2: Quantities lifting gate

#	No.	Diameter	Thickness	Section	Length	Volume	Weight	Weight tot
		[mm]	[mm]	[mm^2]	[m]	[m^3]	[ton]	[ton]
1	2	711	30	67010	53	3.5	27	55
2	28	219	16	11008	14	0.2	1	34
3	12	406	25	31887	6	0.2	1	18
4	24	219	16	11008	14	0.2	1	29
5	29		20	12000	53	0.6	5	140
6	26		25	25000	20	0.5	4	102
7	2	508	25	39898	53	2.1	16	33
8	1		20	400000	53	21.0	164	164
9	2			300000	20	6.0	47	94
								669

The preliminary design confirms the total mass of each gate of approx. 700 tonnes

4.3 Number of gates

The discharge capacity required to keep the water level in the reservoir below 5 m MSL is 110,000 m^3/s. This value is higher than the incoming peak flow of about 86,000 m^3/s since water cannot always be discharged; only when water level difference between the reservoir level and tidal level is larger than 0.2 m the gates will be open. This discharge capacity is translated into a spillway design that consists of 35 spans of 50 m each separated by 34 piers of 6 m thick, resulting in a net width of 1750 m and a gross width of 1954 m.

4.4 Cost estimate for gate system

The following cost estimate is derived based on the conclusions above. i.e. 35 no. gates of 700 tonnes each.

Table 4-3: Cost Estimate for Gate System

No.	Item Description	Unit	Quantity	Rate	Amount
1	Supply of steel	tonne	24,500	INR 90,000	INR 2,205,000,000
2	Fabrication and installation of gates	% of steel supply amount		3	INR 6,615,000,000
3	**Subtotal Gates (1+2)**				**INR 8,820,000,000**
4	Supply and installation of operating mechanism and mechanical ancillaries	% of gate supply, fabrication and installation amount		0.3	INR 2,646,000,000
5	**Subtotal operating Mechanism (3)**				**INR 2,646,000,000**
6	**TOTAL GATES + OPERATING MECHANISM (3+5)**				**INR 11,466,000,000**

Notes:
1. The supply price of steel is currently extremely volatile. The rate used herein is based on long-term trends and thus ignores the current steep spike in price.
2. The estimate above is a current estimate and thus does not take into account escalation in rates between the current date and date of construction.
3. This cost estimate excludes the cost of the civil structures (reinforced concrete, etc) associated with the spillway and gates, and excludes any applicable taxes.

5 Spillway gates - operation and maintenance (O&M) plan

5.1 General information related to O&M costs

A general rule of thumb is that for hydromechanical gate systems the yearly maintenance and inspection cost is approximately 1% of the investment cost.

The replacement intervals for the various components of the system are as shown below. It is stressed that the replacement intervals and maintenance intervals are related to availability requirements.

Table 5-1: Maintenance Intervals

Part	Measure	Intervention level	Replacement Interval
Concrete structures	Replace concrete cover	% visible damage	25-35 yr
Gate	Paint replacement Replace gates	% visible damage	10-15 yr 50-60 yr
Hydraulic structures	Replace systems	End of life time, economical considerations	20-30yr
Information	Replace safety systems, communication, etc	End life time, economical considerations	8-12 yr
Energy supplies	Emergency systems Normal systems		15-30 yr 25-30yr
Operation Building(s)	Maintenance Replacement		10 yr 50 yr

In general it can be concluded that most of the inspection and maintenance effort for these kinds of civil structures is focussed on the operating systems and hydraulic systems. Maintenance of the civil parts is comparatively minor and predictable. For example, painting of the truss structure is required every 10 years. For the smaller elements it is not possible to specify the maintenance or replacement period, as this highly depends on the type of elements and its contribution to the failure probability.

5.2 Operation of spillway gates

5.2.1 General assumptions of operating system

The vertical lifting gates of the spillway are operated by a hydraulic installation. Hydraulic cylinders are provided on each side of the gate for lifting and lowering. The two cylinders with trunnion connection and cardan suspension are mounted with the piston rod downwards as shown schematically in the figure below.

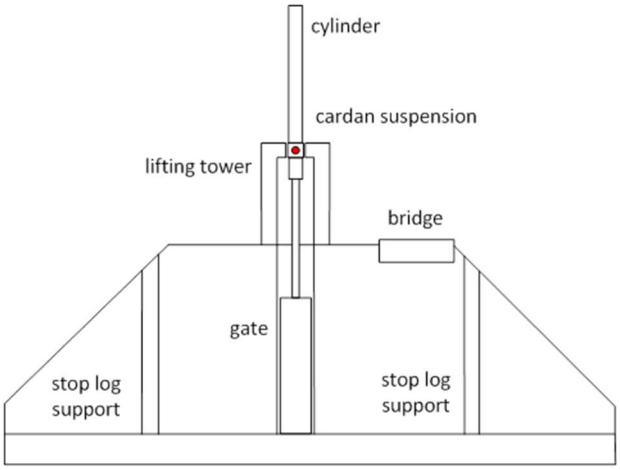


Figure 5-1: Section through gate arrangement

For the purpose of measuring the retracted length of the cylinder, the cylinders are equipped with an integrated position detection system. As backup for the integrated system and for means of control, limit switches and an inclinometer are provided on the gates. An emergency power system will be provided for monitoring of the position detection system in case of power failure.

Hydraulic units connected to the cylinders deliver pressurised oil to the cylinders in order to operate the gate. A so-called 'open' system will be used. Each cylinder is connected to its own hydraulic-unit. The hydraulic units consist mainly of an oil-tank, two pumps with adjustable stroke volume behalf of the speed control, an emergency pump which will be independent of the normal control system and normal energy-connection, a valve block with the required protections, valves, directional control valves and the required instrumentation. The units are further provided with a bellow to collect the ousted air from the oil-tank when filled by the shuttle volume of the cylinder. The bellow will be placed in a protecting steel cover.

5.2.2 Operations

Starting points

- The number of corrections of the gate will be limited to one position correction an hour.
- The tolerance on the requested set-point of the gate will be 10 mm.
- After motion, the position of the gate will be fixated by the (fail safe) brake valves on the valve block on the cylinders
- In case breakage of a component in the oil supply to the cylinder (under pressure), the position of the gate will be fixated with the (fail safe) brake valves on the cylinders, preventing the gate uncontrolled lowering.
- The both cylinders for the gate operation will operate in a so called 'master/slave' configuration
- The deceleration of the gate will be checked with a retard-protection.
- The gates work independently from each-other.
- For maintenance actions to the gate, the gate must be switched to a maintenance position.

Normal operation

- The required position of the gates will be determined by the control system.
- Depending on the set-point and the actual position of the gate, the control system determines the motion strategy to follow: 'minor correction' or 'normal movement'.

Minor correction:

- After receiving the set-point, only one main pump (the pumps will be stated alternately) will be started. The pump will start pressure less.

- When the motor is started and on nominal speed, the gate will be started with creepage speed during 6 seconds.
- After the creep speed the gate will unfirmly accelerate to a given velocity (set-point, about 1/10 of the maximum velocity with two pumps) in 4 seconds.
- When the position of the gate is near to the set-point, the gate will retard with a uniform deceleration in 4 seconds to creep speed.
- The motion will proceed in creep speed and will be stopped when reaching the set-point
- When stopped, the position of the gate will be fixated with the brake valve (with safe detection) at the cylinder valve block.
- The pump will be stopped.

Normal movement:

- After receiving the set-point, the main pumps of the hydraulic-units on both sides of the gate will be started. The pumps will start pressure less.
- When the motors are started and are on nominal speed, the motion of the gate up will be started with creepage speed during 6 seconds.
- after the creepage speed the gate will uniformly accelerate to maximum motion velocity in 12 seconds.
- The gate will move up- or downwards with uniform velocity.
- When the position of the gate is near to the set-point or the ‘pre-open’ or ‘pre-closed’ postion of the gate is detected, the gate will retard with a uniform deceleration in 12 seconds to creepage speed.
- The motion will proceed in creep speed and will be stopped when reaching the set-point tor the position ‘open’ or ‘closed’.
- When stopped, the position of the gate will be fixated with the brake valve (with safe detection) at the cylinder valve block.
- The pumps will be stopped.

Synchronous motion of the cylinders

With the position detection system, the retracted length of the cylinders and thereby the open position of the gates is known. The position detection system will be used for synchronous motion of the cylinders, to prevent misalignment and blockage of the gate. The cylinders will be electronically connected to the control system, which will regulate synchronous movement of the cylinders by reducing the pump capacity of the lagging cylinder. In case the inclination of the gate still incurs, the motion of the gate will be stopped.

Check on leakage

The control system will also use the position detection system to maintain the open position of the gate. In case the position of the gate changes too much, because of for example some internal leakage in the hydraulic system, the control system will correct the position of the gate. In case the number of corrections is too high an alarm signal will be given.

Check on position-detection

To check the position detection on malfunction, the gate is provided with an inclinometer and limit switches. When the inclinometer detects inclination of the gate while the position detection system detects synchronous motion an alarm signal will be given and the motion of the gate will be stopped. limit switches will be used as back-up system for the end positions of the gate.

Emergency operation

In case of failure of the control system, the gates can be operated by using the local controls panel in the machine room. To be able to move the gate in straight position the output value of the position detection systems of the gate and the inclinometer are presented on a display near to the control panel. Inclination of the gates has to be corrected manually by stopping shortly the motion of the lagging cylinder.

Emergency stop
In case of an emergency stop, the gate will be deaccelerating uniformly to zero in 6 seconds.

Retard protection
To check deceleration of the gate after the position ‘pre-open’ or ‘pre-closed’ an additional limit switch will be placed between ‘pre-open’ and ‘open’ and between ‘pre-closed’ and ‘closed’. After detecting ‘pre-open’ a timer will start. In case the position ‘open’ will be reached sooner than expected this means the gate didn’t decelerate and an emergency stop will occur to protect the gate and the hydraulic operation mechanism for damage. The same principle will be used for the motion down wards. The extra limit-switches are provided because the position detection system in the cylinder is not be to obtain in safety class SIL-2.

5.2.3 Interface with the control system

Besides the necessary interaction between the control and the hydraulic system for the correct movement of the gate, there will be additional malfunction alarms, at least the following detections and protections are provided:

Detection hydraulic unit
- low oil level in the oil tank;
- too low oil level in the oil tank;
- too high oil temperature;
- too high and too low system pressure;
- dirty filter alarm (bypass filter unit and return oil);
- hydraulic leak detection;
- running hour meters for the main pumps.

Detection instrumentation
- failure of the position system
- failure of the UPS system for the position detection system
- failure of the inclinometer

Protections
- on low oil maximum 24 hour operation;
- on too low oil stop of the gate movement;
- protection against unexpected lowering movement of the gate (by position measuring systems, leak detection);
- on failure of a main pump, to complete the gate movement with the remaining pumps on adapted velocity, but no new movement of the gate will be possible without detection of the failure and release of the operation with reduced velocity.

5.2.4 Maintenance and inspection operating system

For proper function of the gates following maintenance activities will be needed:

Table 5-2: Maintenance activities for gates

Description	Inspection, test and minor repairs	Expected life span	Maintenance
	(x/year)	(year)	
Hydraulic unit with oil tank	6	25	The hydraulic unit should visual inspected 6 times a year and twice a year be cleaned properly. Oil leakages should be recovered. Once a year the operation pressures should be checked and compared with the reference analysis. The valve block should be inspected each year
Cylinder	4	50	Visual inspection and check and repaire of leakages and damage. Lubricating the bearing in the rod-eye 4 times year. The valve-block on the cylinder should be inpected and tested each year. After 25 years overhauling of the cylinder and replacement of seals and the valve block.
Connecting pipework	1	25	Check connection pipework and connections for leaks and repair oil leaks.
Oil	1	10	Once a year an oil sample and laboratory check for suspected replacement
Bellow for the shuttle volume of air of the oil tank	2	10	Check of the bellow and the connecting pipework on leakage
Filters	1	5	The filters are equipped with a dirty filter detection. In case of foul filter, the filter should be replaced. Each 5 years the filters elements should be replaced preventively
Hydraulic hoses	1	8	The hydraulic hoses should be inspected each year and replaced preventively after each 8 years
Instrumentation of the hydraulic system	1	25	Yearly functional test. Instrumentation parts should be sufficiently kept in spare and will be exchanged after outage
Switching material	1	12	Annual visual inspection and cleaning contaminants
Cardan suspension	4	50	Visual inspection and cleaning of the construction. The bearings of the cardan suspension should be lubricated 4 times a year. Annual check and repair of damages

Description	Inspection, test and minor repairs	Expected life span	Maintenance
	(x/year)	(year)	
Hydraulic unit with oil tank	6	25	The hydraulic unit should visual inspected 6 times a year and twice a year be cleaned properly. Oil leakages should be recovered. Once a year the operation pressures should be checked and compared with the reference analysis. The valve block should be inspected each year
Cylinder	4	50	Visual inspection and check and repaire of leakages and damage. Lubricating the bearings in the rod-eyes 4 times year. The valve-block on the cylinder should be inpected and tested each year. After 25 years overhauling of the cylinder and replacement of seals and the valve block.
Connecting pipework	1	25	Check connection pipework and connections for leaks and repair oil leaks.
Oil	1	10	Once a year an oil sample and laboratory check for suspected replacement
Bellow for the shuttle volume of air of the oil tank	2	10	Check of the bellow and the connecting pipework on leakage
Filters	1	5	The filters are equipped with a dirty filter detection. In case of foul filter, the filter should be replaced . Each 5 years the filters elements should be replaced preventively
Hydraulic hoses	1	8	The hydraulic hoses should be inspected each year and replaced preventively after 8 years
Instrumentation of the hydraulic system	1	25	Yearly functional test. Instrumentation parts should be sufficiently kept in spare and will be exchanged after outage
Switching material	1	12	Annual visual inspection and cleaning contaminants
Sheave struction motion cables and counterweight cables	4	50	Visual inspection and cleaning of the construction. The bearings should be lubricated 4 times a year. Anual check and repair of damages
cablework (motion and counterweight)	2	15	Cleaning and lubricating 2 times a year. Annual technical inspection of the cables on damage
operation mechanisme leveling gates	4	15	Visual inspection and cleaning of the construction. The bearings should be lubricated 4 times a year. Anual check and repair of damages

6 Spillway gates - risks and future considerations

6.1 Risks

The main risks related to the broader context of the spillway design wherein the spillway gates are to be located are as follows:

- Type of foundation. A shallow foundation or a needed pile foundation, depending on the type of soil have a considerable consequence related to the total costs of the spillway structure. A detail soil investigation is essential.
- Design conditions related to amount of discharge of water, time periods will determine the final width of the spillway and will have a considerable impact on the total costs of the structure.
- The feasibility of constructing the spillway. A detail study is essential to determine what is possible how to build the spillway.

6.2 Future Considerations

The Kalpasar project construction procurement strategy will have a significant impact on how the design of the lifting gates at the spillway will be taken forward. For large hydromechanical elements such as these gates, it is standard international practice for the gates to be provided through a so-called design and build (D&B) contract, where the client provides a performance specification for the gates to the prospective tenderers. However other procurement approaches could also be pursued. The procurement strategy chosen will dictate what future steps need to be taken with regards to the spillway gates.

In any event, before the gates are fabricated and installed, the contracted party would need to perform detailed design of the structure and steel profiles to optimize material use and ensure robustness (including inter alia consideration of vibration-induced fatigue, check for buoyancy, confirmation and assessment of loading criteria including earthquake loads, steel coating selection, detailing of interface between steel and concrete, etc), all within the framework of the type of contract selected for use.

7 Spillway gates - conclusions

A multi-criteria analysis of the gate type options for the spillway gates was performed, and the vertical lifting gate type was selected as being most favourable.

A multi-criteria analysis of the preferred gate operational mechanism for the vertical lifting gates was then undertaken, which concluded that the electromechanical system with two winches (Variant C) is preferred.

A comparison was undertaken between lens-shaped and flat-shaped vertical lifting gates, concluding that lens-shaped gates are preferred in this instance.

A conceptual design of the lens-shaped lifting gate has been described, steel quantities calculated and a cost estimate determined for the lens-shaped vertical lifting gates which could be applied in the discharge sluices of the Kalpasar Dam.

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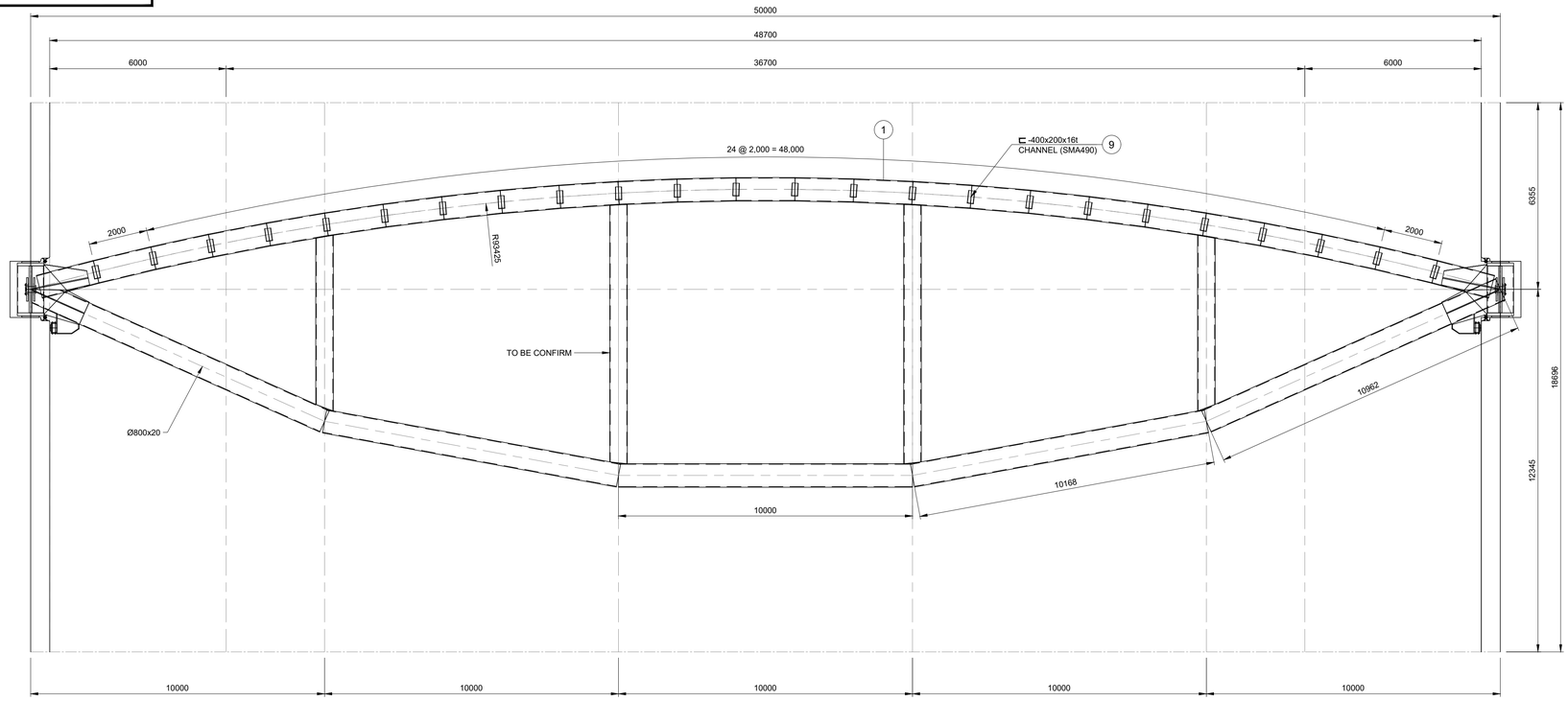
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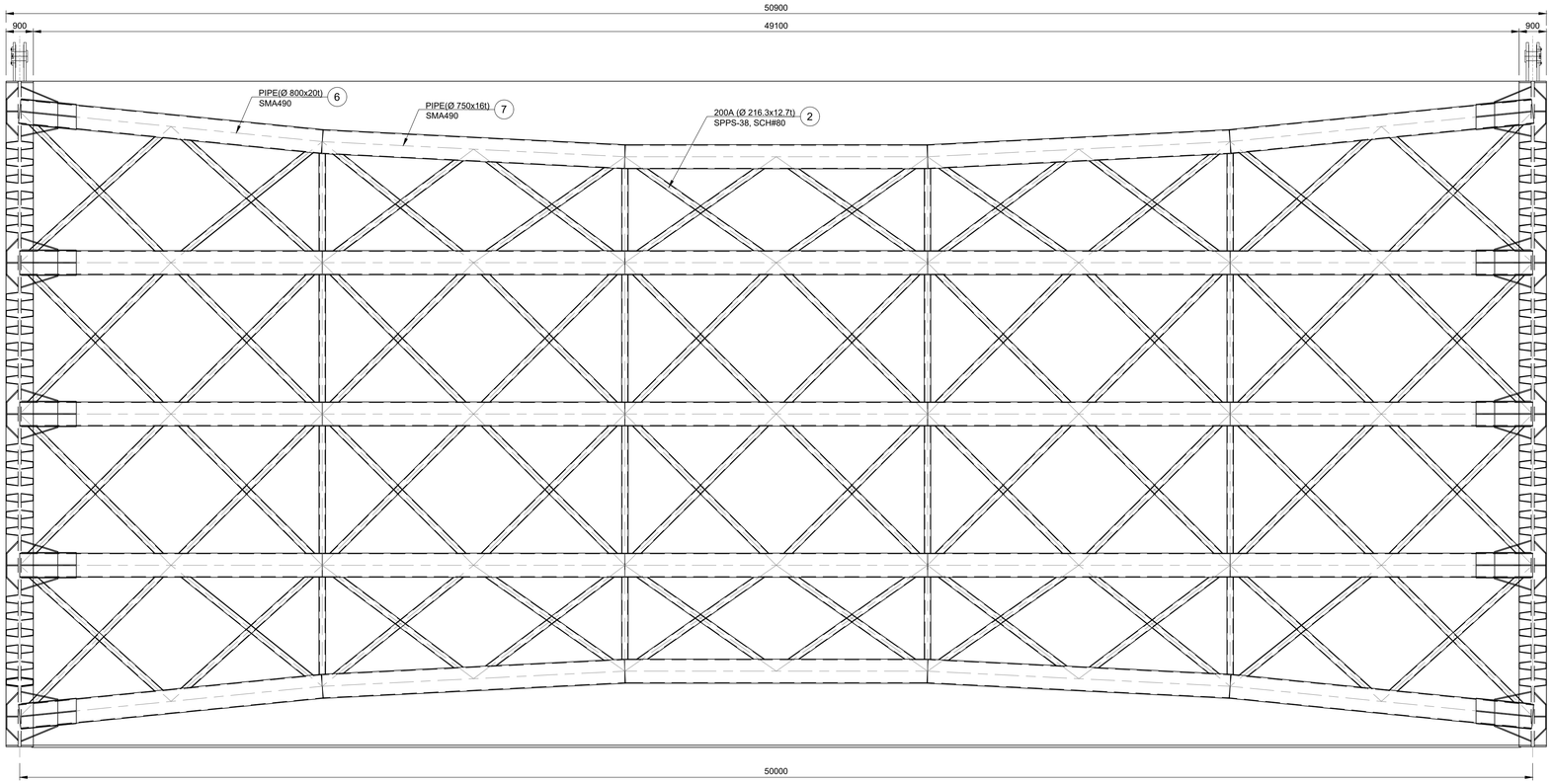
APPENDIX A: DESIGN DRAWINGS

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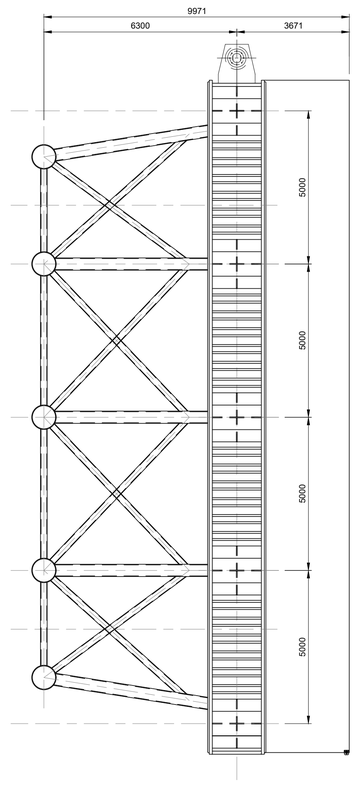
DO NOT SCALE



VIEW A-A (OPTION - 2)
SCALE 1:100



ELEVATION
SCALE 1:100



VIEW E-E
SCALE 1:100

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NOTES

1. ALL DIMENSIONS ARE IN MILLIMETRES, UNLESS NOTED OTHERWISE.
2. ALL LEVELS ARE IN METRES ABOVE CHART DATUM (MARKED ±0.00m CD).
3. ANY ELEMENTS OR OBJECTS THAT ARE NOT DIMENSIONED SHALL NOT BE SCALED UNDER ANY CIRCUMSTANCES. THEY SHALL BE AGREED/CONFIRMED WITH THE EMPLOYER / ENGINEER.
4. THE MEMBER SIZING ARE INDICATIVE AND THEY SHALL BE FURTHER REFINED IN DETAIL DESIGN STAGES.

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REVISIONS

CLIENT
NATIONAL CENTER FOR COASTAL RESEARCH

PROJECT
DPR FOR DAM CLOSURE AND SPILLWAY GATE DESIGN FOR KALPASAR DAM

TITLE
TYPICAL SPILLWAY GATE - KALPASAR

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DRAWING No.	SUITABILITY	REVISION
DI1578-RH-XX-ZZ-DR-C-0001	A1	P01